

DESIGN OF EARTH-QUAKE RESISTANT MULTI-STORIED RCC BUILDING ON A SLOPING GROUND

A THESIS SUBMITTED IN PARTIAL FULFILLMENT
OF THE REQUIREMENTS FOR THE DEGREE OF
BACHELOR OF TECHNOLOGY

IN
CIVIL ENGINEERING
BY
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Under the Guidance of
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Department of Civil Engineering
National Institute of Technology
Rourkela
2010

CANDIDATES DECLARATION

I hereby declare that the work which is being presented in this project titled “**DESIGN OF EARTH-QUAKE RESISTANT MULTI-STORIED RCC BUILDING ON A SLOPING GROUND**” for partial fulfilment of the requirements for the award of degree of **BACHELOR OF TECHNOLOGY** in **CIVIL ENGINEERING** submitted in the **NATIONAL INSTITUTE OF TECHNOLOGY,ROURKELA** is an authentication record of my work carried during the period from may 2009 under the supervision of **DR.K.C.Biswal**

Date:

Place:

SAPTADIP SARKAR
Department of Civil Engg.
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Rourkela

**NATIONAL INSTITUTE OF TECHNOLOGY
ROURKELA**

CERTIFICATE

This is to certify that the thesis entitled, Design of **DESIGN OF EARTH-QUAKE RESISTANT MULTI-STORIED BUILDING ON A SLOPING GROUND** submitted by **MR.SAPTADIP SARKAR** in partial fulfilment of the requirements for the award of Bachelor of technology in **Civil Engineering** at the **National Institute of Technology, Rourkela** is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University/Institute for the award of any degree.

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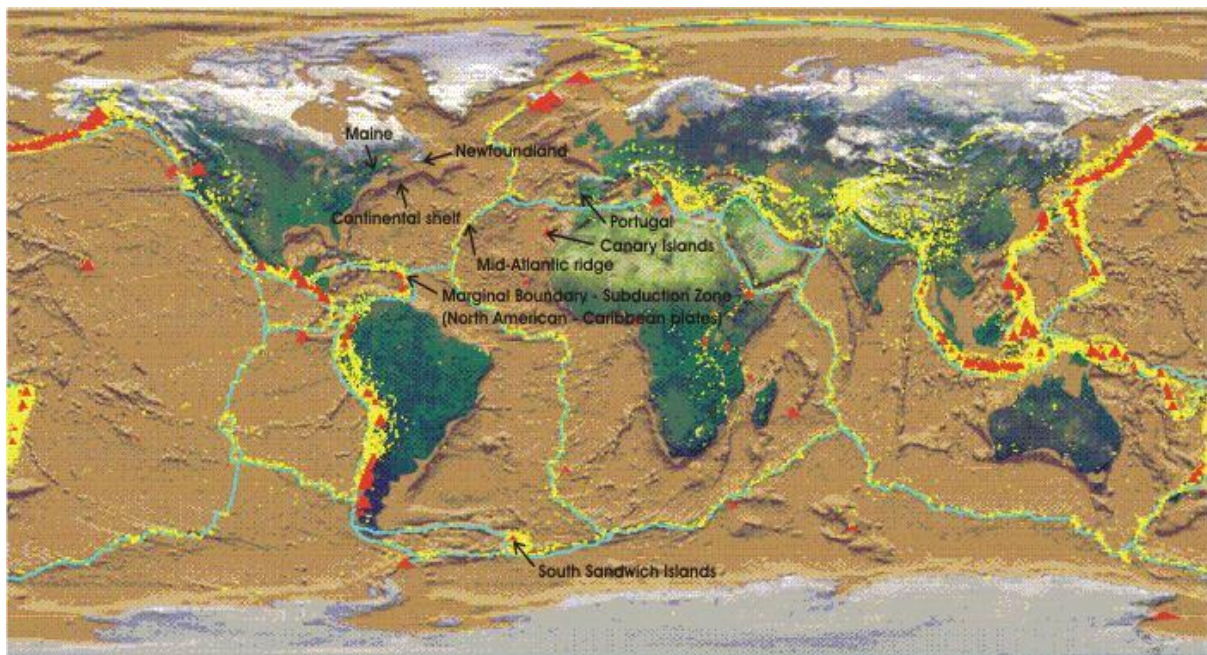
Abstract

This project named as “**DESIGN OF EARTH-QUAKE RESISTANT MULTI-STORIED RCC BUILDING ON A SLOPING GROUND**” involves the analysis of simple 2-D frames of varying floor heights and varying no of bays using a very popular software tool STAAD Pro. Using the analysis results various graphs were drawn between the maximum axial force, maximum shear force, maximum bending moment, maximum tensile force and maximum compressive stress being developed for the frames on plane ground and sloping ground. The graphs used to drawn comparison between the two cases and the detailed study of “**SHORT COLOUMN EFFECT**” failure was carried up. In addition to that the detailed study of seismology was undertaken and the feasibility of the software tool to be used was also checked. Till date many such projects have been undertaken on this very topic but the analysis were generally done for the static loads i.e. dead load, live load etc, but to this the earthquake analysis or seismic analysis is to be incorporated. To create a technical knowhow, two similar categories of structures were analyzed, first on plane ground and another on a sloping ground. Then the results were compared. At last the a structure would be analyzed and designed on sloping ground for all possible load combinations pertaining to IS 456, IS 1893 and IS 13920 manually.

INTRODUCTION

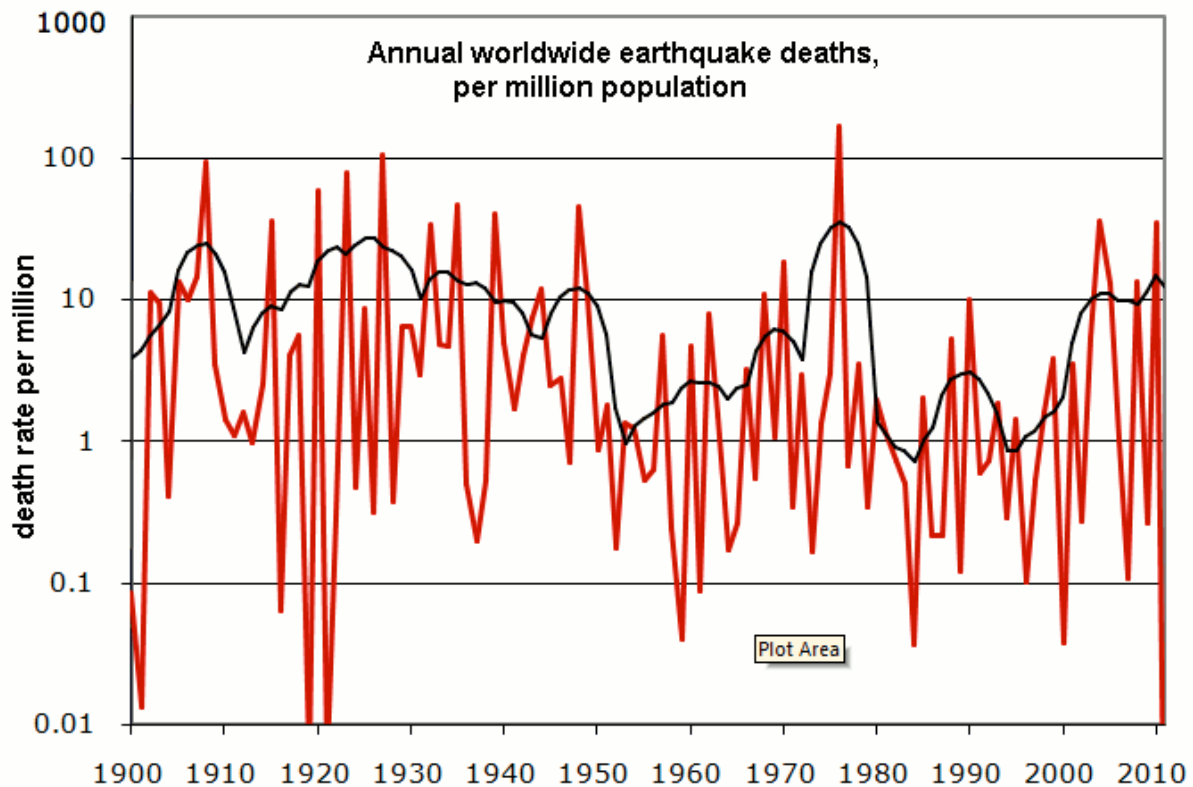
Seismology is the study of vibrations of earth mainly caused by earthquakes. The study of these vibrations by various techniques, understanding the nature and various physical processes that generate them from the major part of the seismology. [2]*

Elastic rebound theory is one such theory, which was able to describe the phenomenon of earthquake occurring along the fault lines. Seismology as such is still a very unknown field of study where a lot of things are yet to be discovered. [2]*



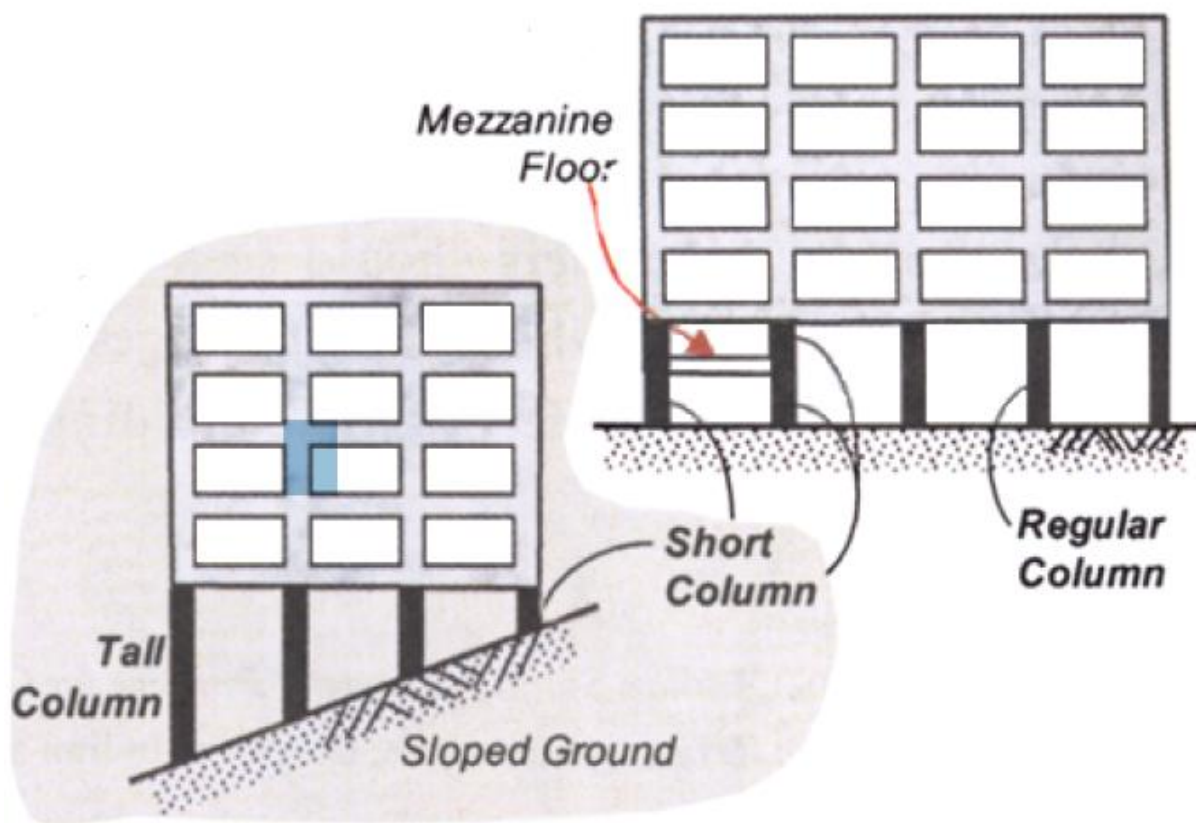
The above Picture is showing the fault lines and we can see that epicentres are all concentrated all along the fault lines. The reason for seismic activities occurring at places other than the fault lines are still a big question mark. Also the forecasting of earthquake has not been done yet and would be a landmark if done so.

There is general saying that it's not the earthquake which kills people but its the bad engineering which kills people. With industrialization came the demand of high rise building and came dangers with that.

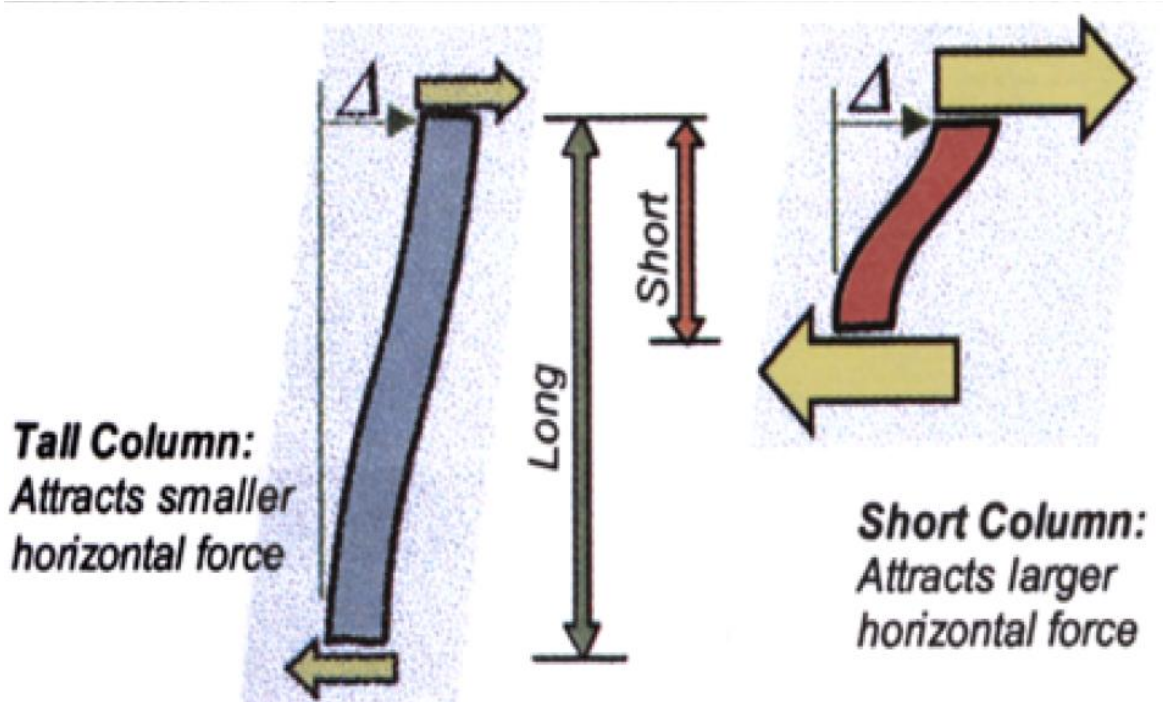


A seismic design of high rise buildings has assumed considerable importance in recent times. In traditional methods adopted based on fundamental mode of the structure and distribution of earthquake forces as static forces at various stories may be adequate for structures of small height subjected to earthquake of very low intensity but as the number of stories increases the seismic design demands more rigorous. [1]*

During past earthquakes, reinforced concrete (RC) frame buildings that have columns of different heights within one storey, suffered more damage in the shorter columns as compared to taller columns in the same storey. Two examples of buildings with short columns in buildings on a sloping ground and buildings with a mezzanine floor can be seen in the figure given below. [1]*



Poor behaviour of short columns is due to the fact that in an earthquake, a tall column and a short column of same cross section move horizontally by same amount which can be seen from the given figure below. [1]*

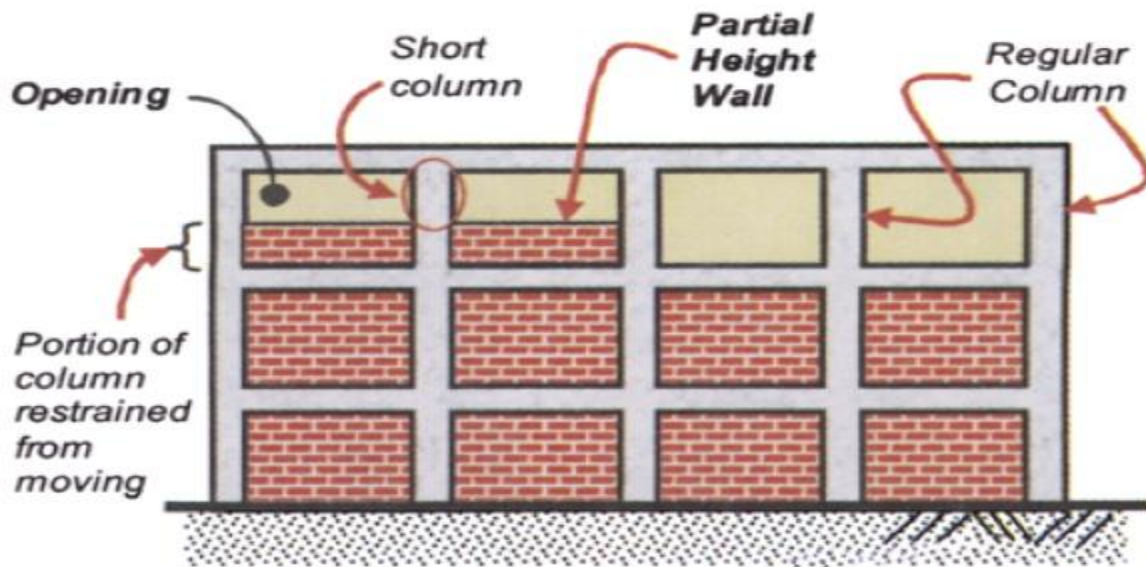


However, the short column is stiffer as compared to the tall column, and it attracts larger earthquake force. Stiffness of a column means resistance to deformation- the larger is the stiffness, larger is the force required to deform it.

If a short column is not adequately designed for such a large force, it can suffer significant damage during an earthquake. This behaviour is called **Short Column Effect**. The damage in these short columns is often in the form of X-shaped cracking - this type of damage of columns is due to **shear failure**.



Many situations with short column effect arise in buildings. When a building is rested on sloped ground, during earthquake shaking all columns move horizontally by the same amount along with the floor slab at a particular level (this is called **rigid floor diaphragm action**). If short and tall columns exist within the same storey level, then the short columns attract several times larger earthquake force and suffer more damage as compared to taller ones. The short column effect also occurs in columns that support mezzanine floors or loft slabs that are added in between two regular floors. [1]*



There is another special situation in buildings when short-column effect occurs. Consider a wall (*masonry* or *RC*) of partial height built to fit a window over the remaining height. The adjacent columns behave as short columns due to presence of these walls. In many cases, other columns in the same storey are of regular height, as there are no walls adjoining them. When the floor slab moves horizontally during an earthquake, the upper ends of these columns undergo the same displacement. However, the stiff walls restrict horizontal movement of the lower portion of a short column, and it deforms by the full amount over *the short height* adjacent to the window opening. On the other hand, regular columns deform over the *full height*. Since the effective height over which a short column can freely bend is small, it offers more resistance to horizontal motion and thereby attracts a larger force as compared to the regular column. As a result, short column sustains more damage. X-cracking in a column adjacent to the walls of partial height. In new buildings, *short column effect* should be avoided to the extent possible during *architectural design* stage itself. When it is not possible to avoid short columns, this effect must be addressed in structural design. The Indian Standard IS:13920-1993 for ductile detailing of RC structures requires special confining reinforcement to be provided over the full *height* of columns that are likely to sustain short column effect. The special confining reinforcement (*i.e.*, closely spaced closed ties) must extend beyond the short column into the columns vertically above and below by a certain distance. In existing buildings with short columns, different retrofit solutions can be employed to avoid damage in future earthquakes. Where walls of partial

height are present, the simplest solution is to close the openings by building a wall of full height - this will eliminate the short column effect. [1]*

PHASES OF THE PROJECT

The project was divided into different phases. They are as follows:-

Phase-1- Comparison of design results of STAAD Pro and same problems done manually.

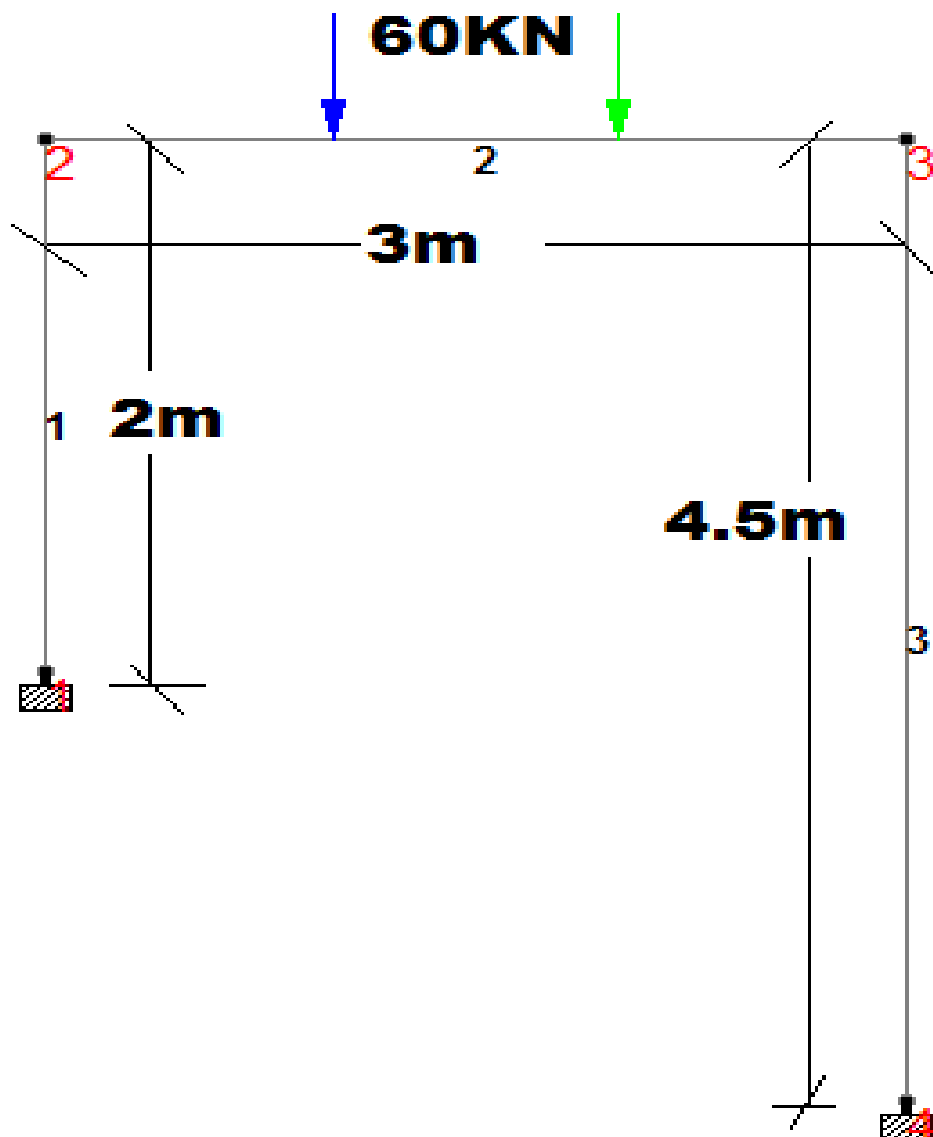
Phase-2- Analysis of simple 2-dimensional reinforced concrete frames and as such both static loads(dead, live and load combinations) and dynamic load (Earth quake load) are to be considered frames first on a plain ground and then on a sloping ground.

Phase-3- Ductility design and detailing.

PHASE 1

Under the 2nd phase we took up a simple 2 dimensional frame subjected to concentrated loads with un-equal supporting columns. This is for the analogy of the actual problem statement of design of a multi-storied building on a sloping ground. [2]*

Problem statement- analysis of a portal frame as shown in the figure:



Solution: we first calculated the distribution factors as given in the table

joint	member	Relative stiffness	Total relative stiffness	Distribution factor
2	2-1(1)	$I/2$	$5I/6$	$3/5$
	2-3(2)	$2I/6$		$2/5$
3	3-2(2)	$3I/9$	$5I/9$	$3/5$
	3-4(3)	$2I/9$		$2/5$

(a) non-sway analysis

Fixed end moments

$$M_{1-2}=M_{2-1}=M_{3-4}=M_{4-3}=0$$

$$M_{2-3} = - [60/3^2] (1 \times 2^2 + 2 \times 1^2) = 40 \text{ kN-m}$$

$$M_{3-2} = +40 \text{ kN-m}$$

	3/5	2/5	3/5	2/5	
0	0	-40	40	0	0
12	24	16	24	-16	-8
3.6	7.2	-12	8	-3.2	-1.60
0.72	1.44	4.80	-4.8	-0.96	-0.48
0.22	0.43	-2.40	2.4	-0.19	-0.10
0.05	0.09	0.96	-1.44	-0.06	-0.03
	0.03	-0.72	0.48	-0.01	
		0.29	-0.29		
		-0.15	0.15		
		0.06	-0.09		
		-0.05	0.03		
		0.02	-0.02		
16.59	33.19	-33.19	+20.42	-20.42	-10.21

So horizontal reaction at 1= $16.59+33.19/2=24.98 \text{ kN}$.

Horizontal reaction at 4= $-20.42-10.21/4.5= -6.8067 \text{ kN}$

Therefore net sway force = $24.89-6.8067= 18.0833 \text{ kN}$.

(b) sway analysis:-

Now the frame will be analysed for a sway force of 18.0833 kN

The ratio of the initial equivalent moments of the column is given by

$$= (I_1/l_1^2) : (I_2/l_2^2) = 20.25 : 4$$

Now the moment distribution is performed choosing the above fixed end moments

	3/5	2/5	3/5	2/5	
-20.25	-20.25	0	0	-4	-4
6.08	12.15	8.10	2.4	1.6	0.80
-0.36	-0.72	1.20	4.05	-1.62	-0.81
+0.36	0.73	-0.48	-2.43	0.10	0.05
-0.02	-0.04	-1.22	-0.24	-0.10	-0.05
0.02	+0.05	0.49	0.14	0.01	0
	0	-1.22	0.25	-0.01	
		0.49	-0.15		
		0.07	-0.02		
		-0.03	0.01		
		0	0.02		
		0	-0.01		
A= -14.17	-8.08	8.08	4.02	-4.02	-4.01

Horizontal reaction at 1= -14.17-8.08/2= -11.125 kN

Horizontal reaction at 4= -4.02-4.01/4.5= - 1.7844 kN

Resolving we have total S=11.125+1.7844 =12.9094 kN

Now the actual sway force of 18.0833 kN , the sway moments will be

18.0833/12.904 x A as in above table.

	1	2	2	3	3	4
A	-14.17	-8.08	8.08	4.02	-4.02	-4.01
Actual sway moment	-19.85	-11.31	11.32	5.63	-5.63	-5.62
Non-sway moments	16.59	33.19	-33.19	20.42	-20.42	-10.21
Final moments	-3.26	21.87	-21.87	26.05	-26.05	-15.83

(c) Reactions

$$H1=9.31 \text{ kN}$$

$$H4=-9.31 \text{ kN}$$

$$V4=61.39 \text{ kN}$$

$$V1=120-61.39=58.61 \text{ kN.}$$

RESULTS OBTAINED FROM STAAD Pro.

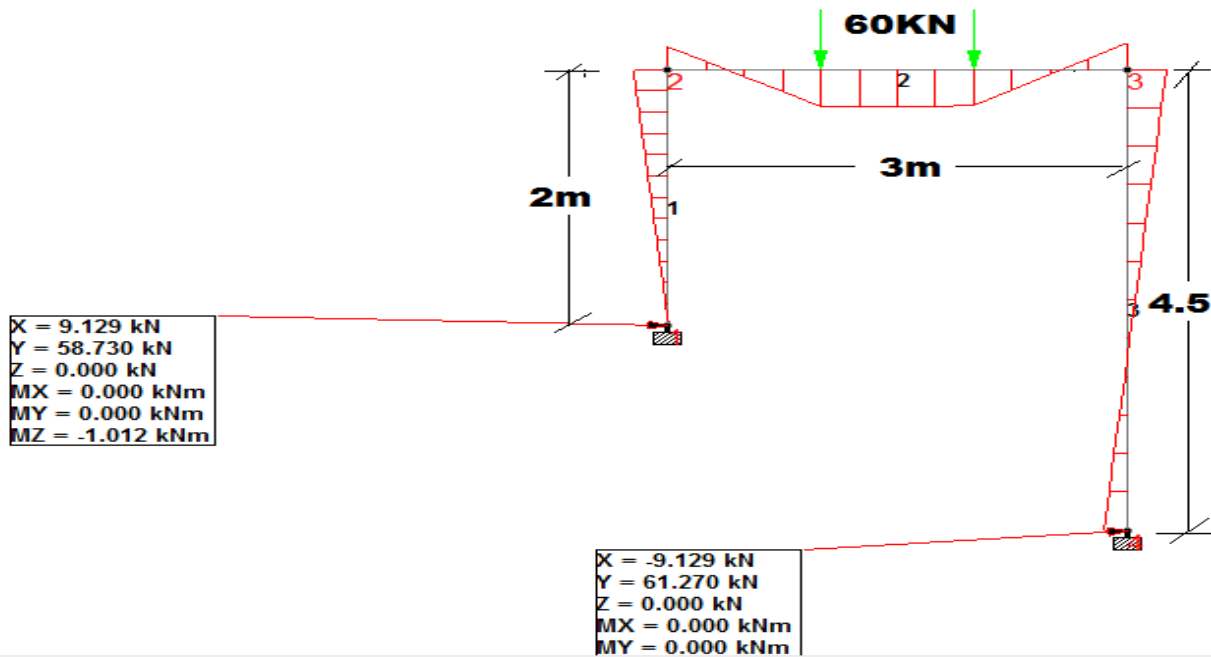
Reactions

Node	L/C	Horizontal	Vertical	Horizontal	Moment		
		FX (kN)	FY (kN)	FZ (kN)	MX (kNm)	MY (kNm)	MZ (kNm)
1	1:concentrated	9.129	58.730	0.000	0.000	0.000	-1.012
4	1:concentrated	-9.129	61.270	0.000	0.000	0.000	15.462

Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kN)	Fy (kN)	Fz (kN)	Mx (kNm)	My (kNm)	Mz (kNm)
1	1	1:CONCENTR	58.864	9.036	0.000	0.000	0.000	-3.959
	2	1:CONCENTR	-58.864	-9.036	0.000	0.000	0.000	22.031
2	2	1:CONCENTR	9.048	58.864	0.000	-100.000	0.000	22.031
	3	1:CONCENTR	-9.048	61.136	0.000	-80.000	0.000	-25.439
3	3	1:CONCENTR	61.136	-9.054	0.000	0.000	0.000	-25.439
	4	1:CONCENTR	-61.136	9.054	0.000	0.000	0.000	-15.304



Final moments (manually) (kN-m)	-3.26	21.87	-21.87	26.05	-26.05	-15.83
From STAAD Pro	-3.959	22.031	-22.031	25.439	-25.439	-15.304

CONCLUSION AT THE END OF PHASE 1-

The results are almost similar and matching with the manual calculations.

PHASE 2

Under phase 2, with full confidence on the STAAD Pro. Design tool, we proceed with the analysis of simple 2 dimensional frames. The analysis was done for both the static load conditions and dynamic load conditions. The 2nd phase involves the analysis of frames on a plane ground and then on a sloping ground.

This 2nd phase can be again broadly divided into following:-

2.1)we first start with 2 storey frame. First we went with double bay and up to 4 bays both on a plane ground and on as sloping ground. we then compare the results.

2.2)we then go for 4 storey frame. For the same we start with double bay and up to 4 bays both on a plane ground and on a sloping ground. we then compare the results

2.3) we then go for 6 storey frame. For the same we start with double bay and up to 4 bays both on a plane ground and on a sloping ground. we then compare the results.

PROBLEM STATEMENT:-

MEMBER PROPERTIES

- All Beams : Rectangular, 400 mm width X 500 mm depth
- All Columns: Rectangular, 400 mm width X 500 mm depth

MEMBER ORIENTATION

- All members : Default

MATERIAL CONSTANTS

- Modulus of Elasticity : 22 KN/sq.mm
- Density : 25 kn/cu.m
- Poisson's Ratio : 0.17

SUPPORTS

- Base of all columns : Fixed

LOADS

Load case 1 : Earth Quake Load

- Zone- III($Z= 0.16$)
- Special revisiting moment frame($RF= 5$)
- Importance factor = 1
- Soil type medium
- RC frame
- Damping ratio=5
- Self weight of the structure.
- 1893 load in global x direction

Load case 2 : Dead Load

- Self weight of the structure.
- Beams : 30 kN/m in global Y downward

Load case 3 : Live Load

- Beams : 200 kN/m in global Y downward

Load Case 4 : DEAD + LIVE

- $L2 \times 1.5 + L3 \times 1.5$

Load Case 5 : DEAD +LIVE+EARTH QUAKE

- $L1 \times 1.2 + L2 \times 1.2 + L3 \times 1.2$

Load Case 6 : DEAD +LIVE-EARTH QUAKE

- $-L1 \times 1.2 + L2 \times 1.2 + L3 \times 1.2$

Load Case 7 : DEAD + EARTH QUAKE

- $L1 \times 1.5 + L2 \times 1.5$

Load Case 8 : DEAD -EARTH QUAKE

- $-L1 \times 1.5 + L2 \times 1.5$

Load Case 9 : DEAD +EARTH QUAKE

- $-L1 \times 1.5 + L2 \times 0.9$

Load Case 10 : DEAD -EARTH QUAKE

- $-L1 \times 1.5 + L2 \times 0.9$

ANALYSIS TYPE : P-DELTA

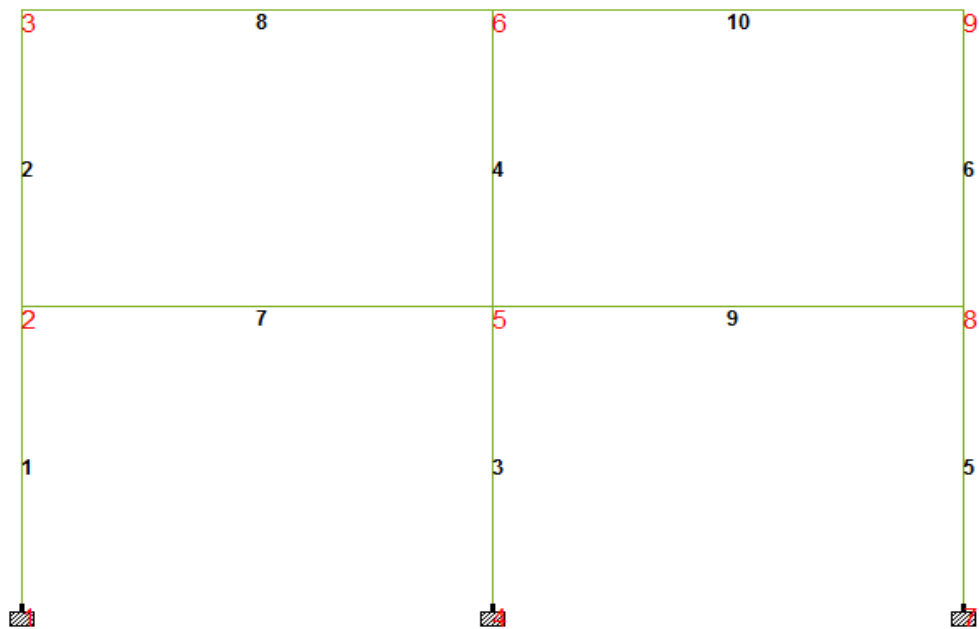
CONCRETE DESIGN:

- Consider all the load cases.
- Parameters: ultimate tensile strength of steel-415 N/sq.mm
- Concrete strength: 30 N/sq.mm
- Clear cover: 30 mm.
- Centre to centre distance of each beam- 4 m
- Height of each storey
 - (a)First the structure is on level ground all the supporting columns being of 4 m height.
 - (b)For the second case the we design the frame for same loading combinations but on a sloping ground of 1 in 5.
- Each beam length = 5m

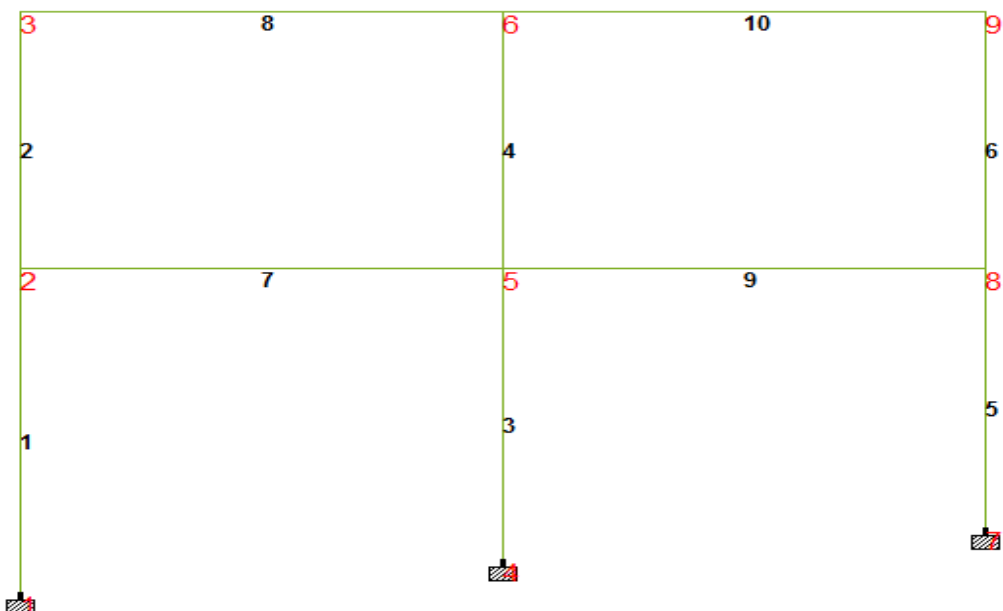
So for this the dimensions of the supporting column are 4m, 4.5 m, 5m, 5.5m and 6m.

PHASE 2.1(DOUBLE BAY)

PLANE GROUND



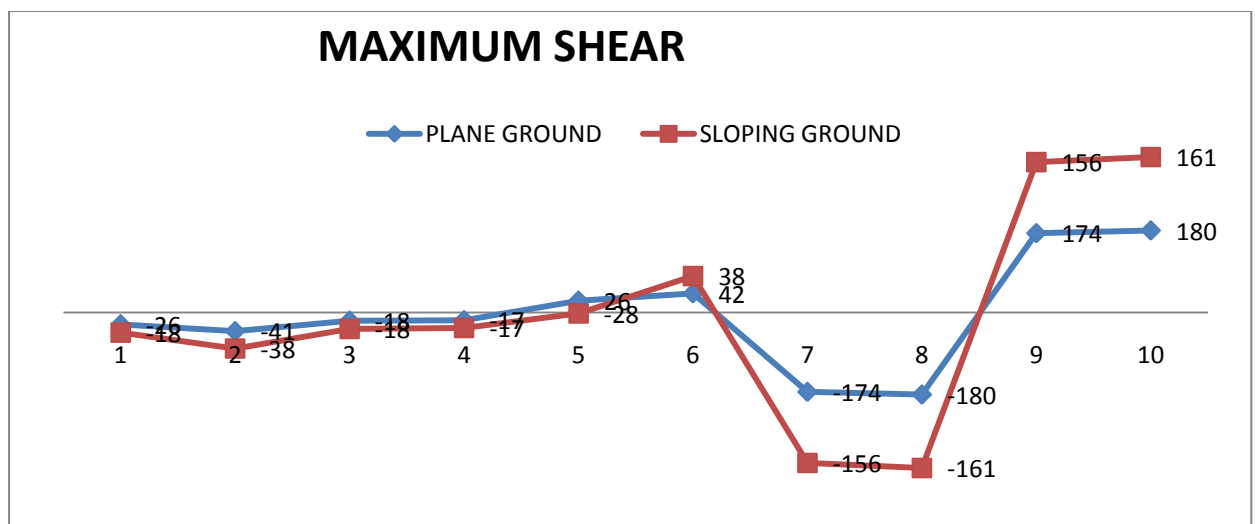
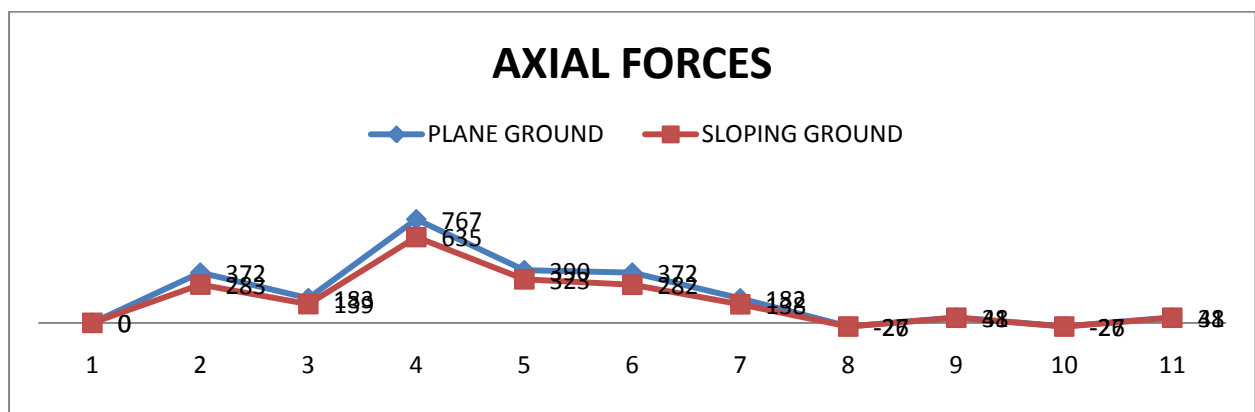
SLOPING GROUND

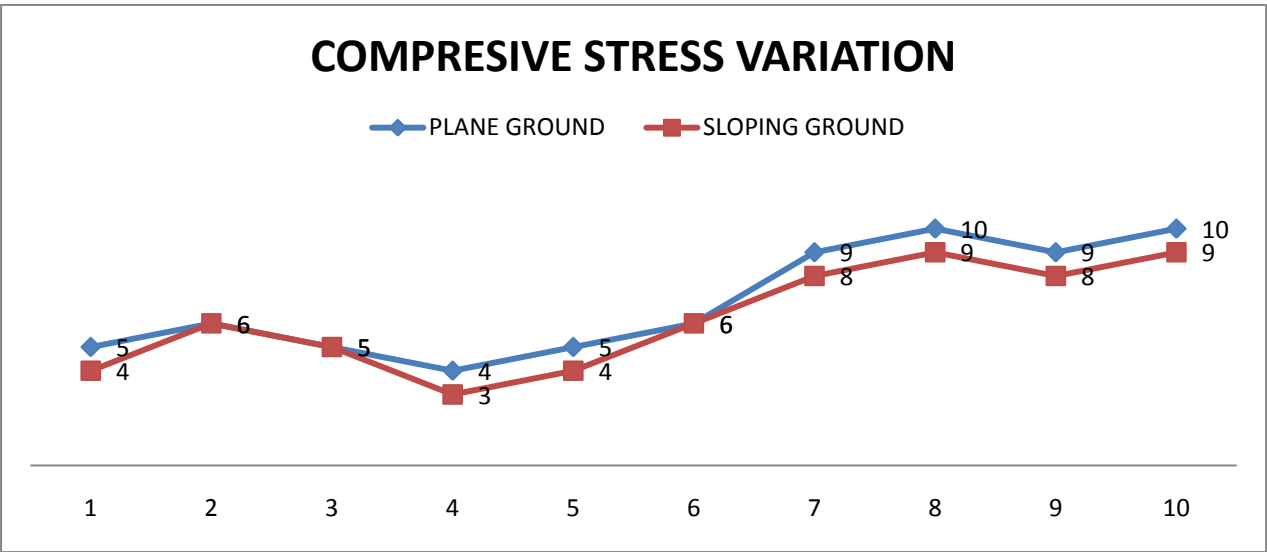
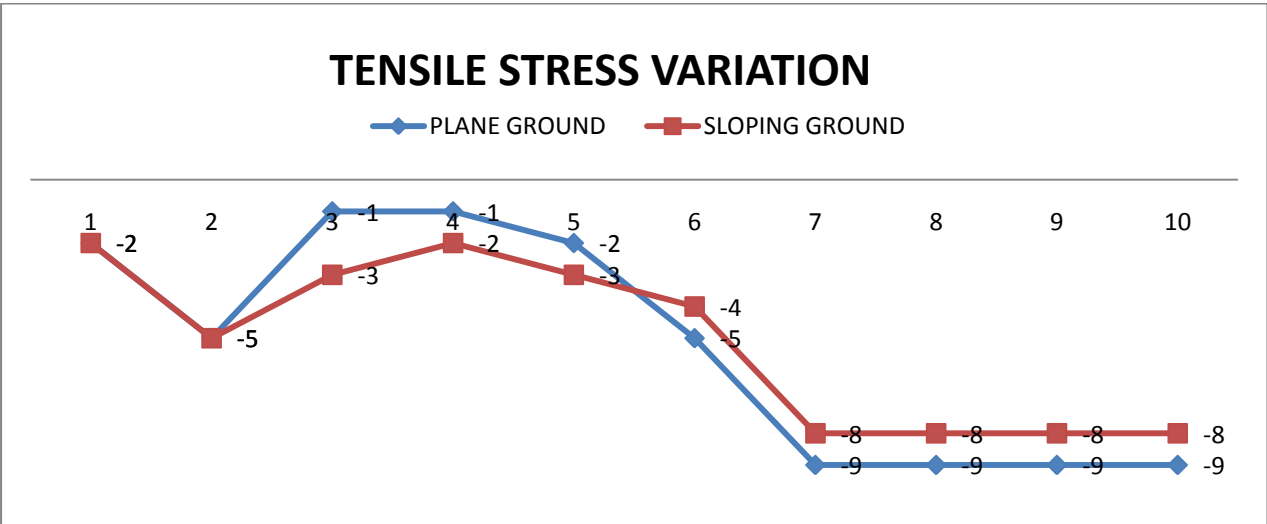
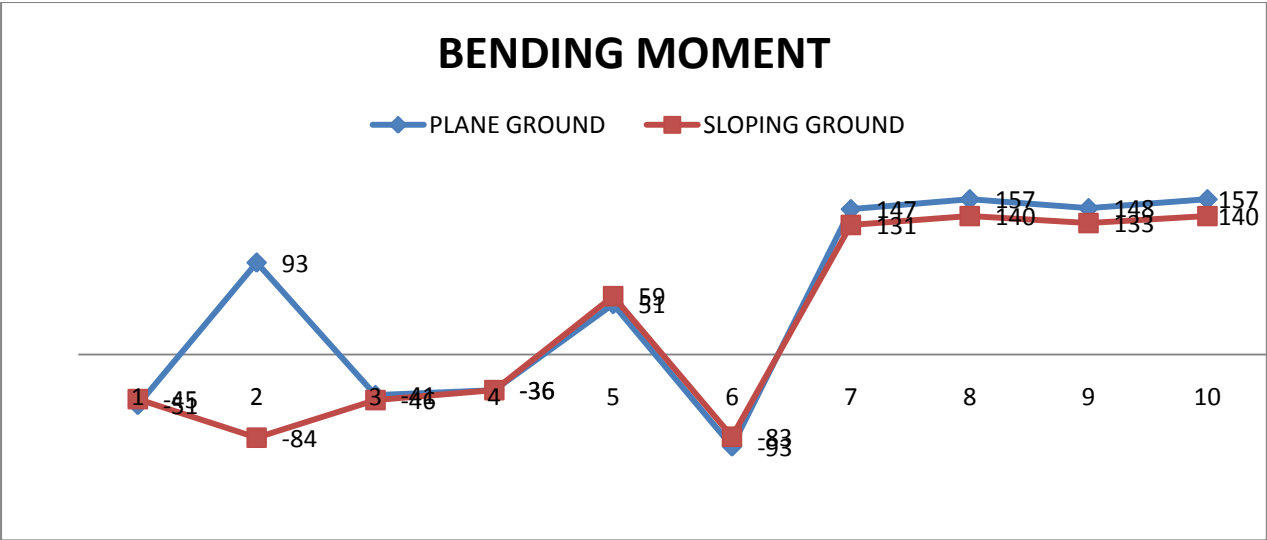


ANALYSIS RESULTS FOR 2 BAY SYSTEMS ON PLANE AND ON A SLOPING GROUND FOR TWO STORY FRAME.

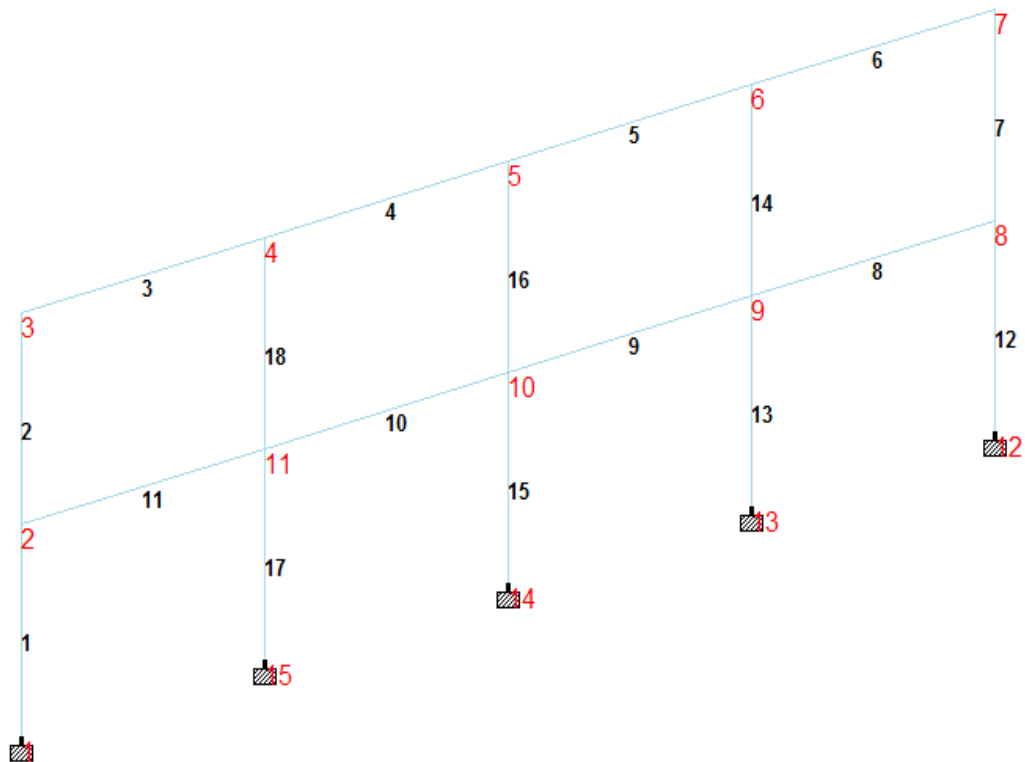
BEAM NO	MAXIMUM AXIAL FORCE		MAXIMUM SHEAR FORCE		MAXIMUM BENDING MOMENT		MAXIMUM TENSILE FORCE		MAXIMUM COMPRESSIVE FORCE	
	kN		kN		kN-m		N/ mm ²		N/ mm ²	
	P*	S*	P	S	P	S	P	S	P	S
1	372	283	-26	-18	-51	-45	-2	-2	5	4
2	183	139	-41	-38	93	-84	-5	-5	6	6
3	767	635	-18	-18	-41	-46	-1	-3	5	5
4	390	323	-17	-17	-36	-36	-1	-2	4	3
5	767	635	26	-28	51	59	-2	-3	5	4
6	182	138	42	38	-93	-83	-5	-4	6	6
7	-26	-27	-174	-156	147	131	-9	-8	9	8
8	41	38	-180	-161	157	140	-9	-8	10	9
9	-26	-27	174	156	148	133	-9	-8	9	8
10	41	38	180	161	157	140	-9	-8	10	9

[P= Plane ground, S= Sloping Ground]*

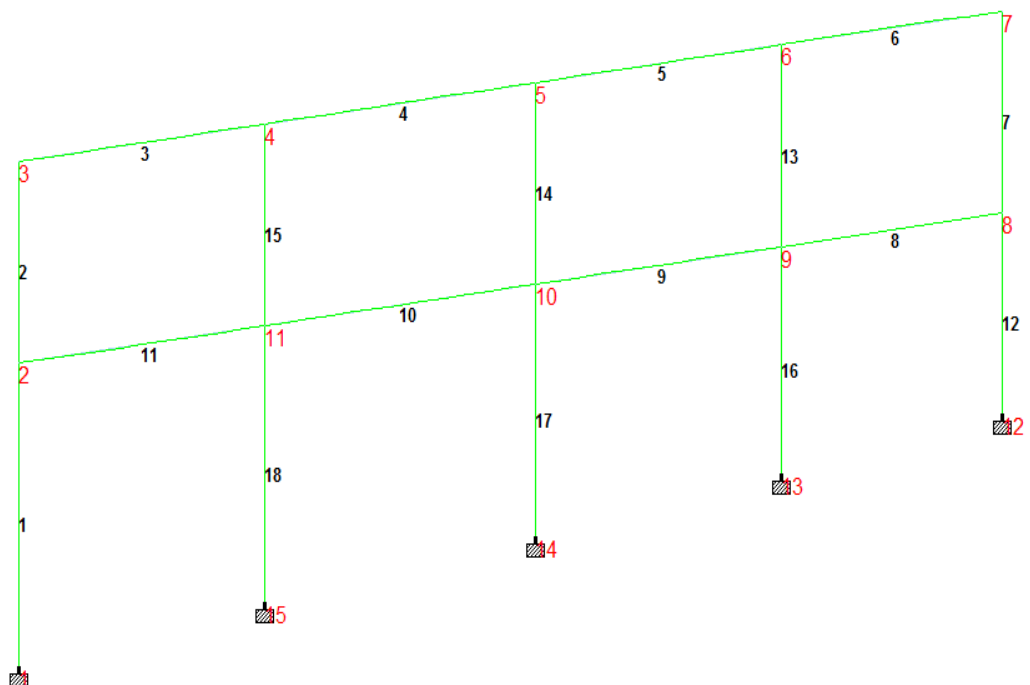




PHASE 2.1(4 BAY SYSTEMS)



PLANE GROUND



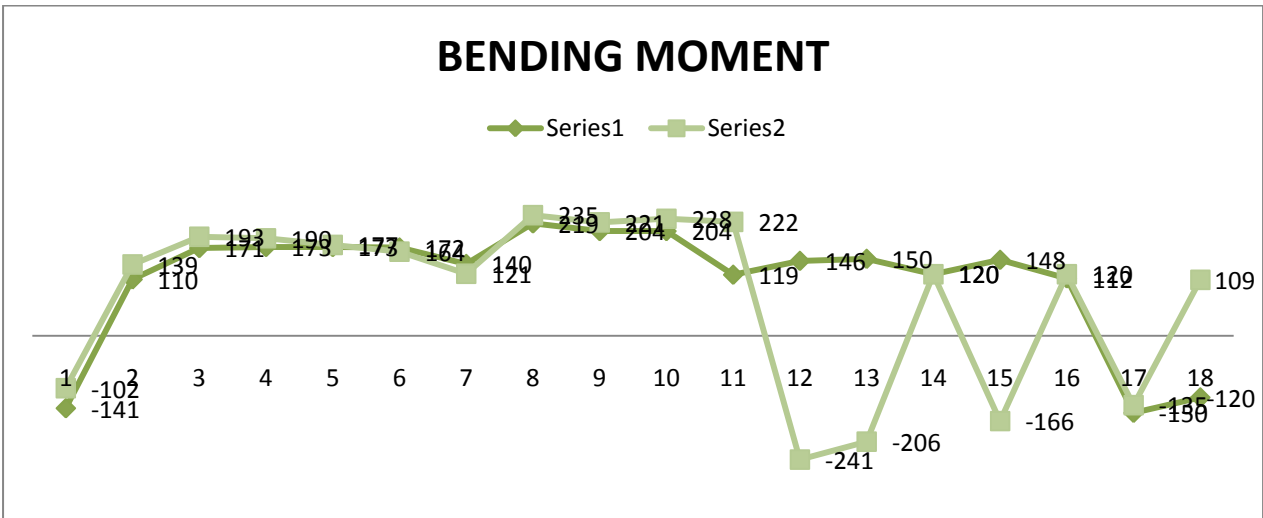
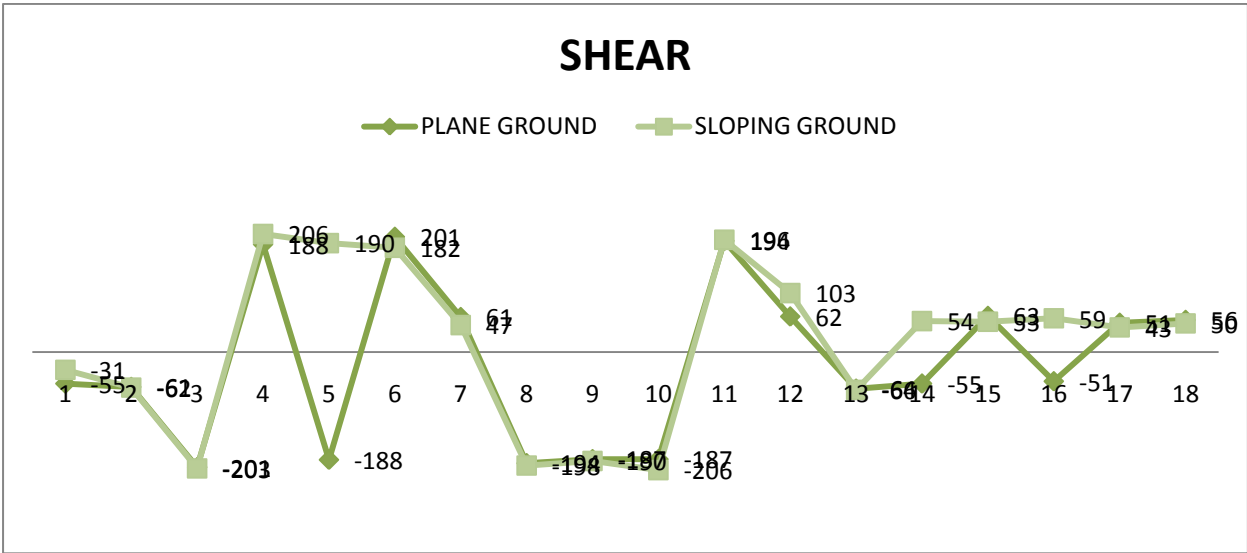
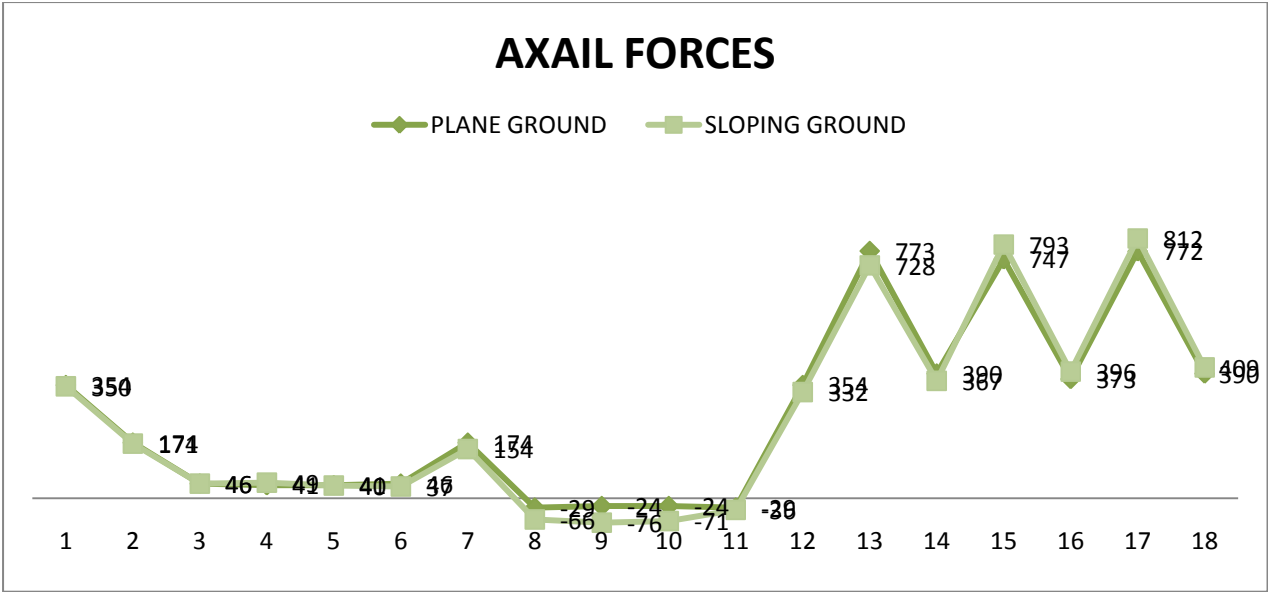
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SLOPING GROUND

ANALYSIS RESULTS FOR 4 BAY SYSTEMS ON PLANE AND ON A SLOPING GROUND FOR TWO STORY FRAME.

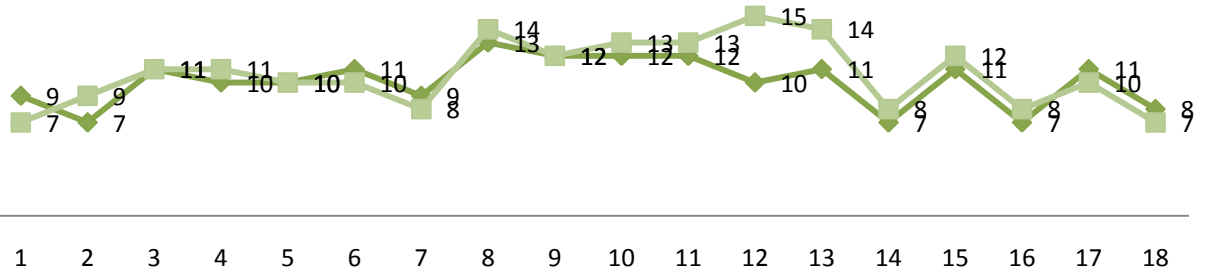
BEAM NO	MAXIMUM AXIAL FORCE		MAXIMUM SHEAR FORCE		MAXIMUM BENDING MOMENT		MAXIMUM TENSILE FORCE		MAXIMUM COMPRESSIVE FORCE	
	kN		kN		kN-m		N/ mm ²		N/ mm ²	
	P*	S*	P	S	P	S	P	S	P	S
1	-7	4	354	350	9	7	-55	-31	-141	-102
2	-7	7	174	171	7	9	-61	-62	110	139
3	11	11	46	46	11	11	-201	-203	171	193
4	10	11	41	49	10	11	188	206	173	190
5	10	10	41	40	10	10	-188	190	173	177
6	11	9	46	37	11	10	201	182	172	164
7	7	6	174	154	9	8	61	47	140	121
8	13	13	-29	-66	13	14	-194	-198	219	235
9	12	13	-24	-76	12	12	-187	-190	204	221
10	12	13	-24	-71	12	13	-187	-206	204	228
11	13	13	-29	-36	12	13	194	+196	119	222
12	7	10	354	332	10	15	62	103	146	-241
13	7	10	773	728	11	14	-64	-66	150	-206
14	6	6	390	367	7	8	-55	54	120	120
15	7	8	747	793	11	12	63	53	148	-166
16	6	6	373	396	7	8	-51	59	112	120
17	7	7	772	812	11	10	51	43	-150	-135
18	6	5	390	409	8	7	56	50	-120	109

[P= Plane ground, S= Sloping Ground]*



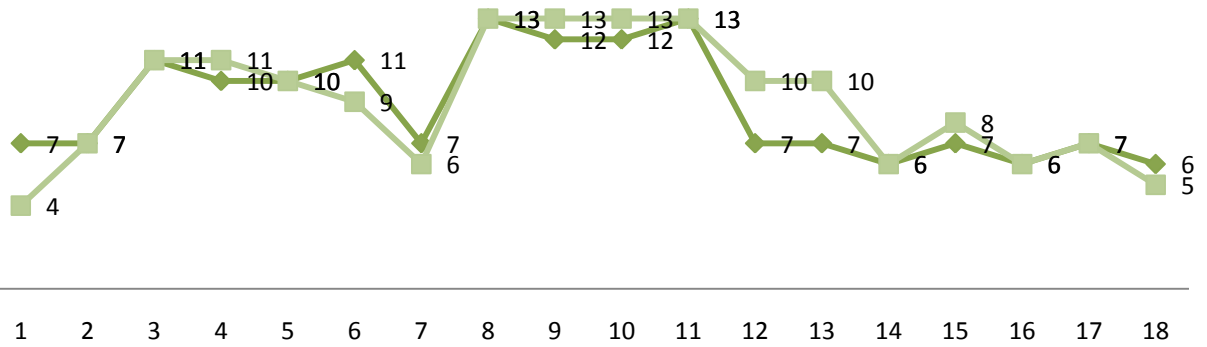
COMPRESSIVE STRESS

◆ PLANE GROUND ■ SLOPING GROUND

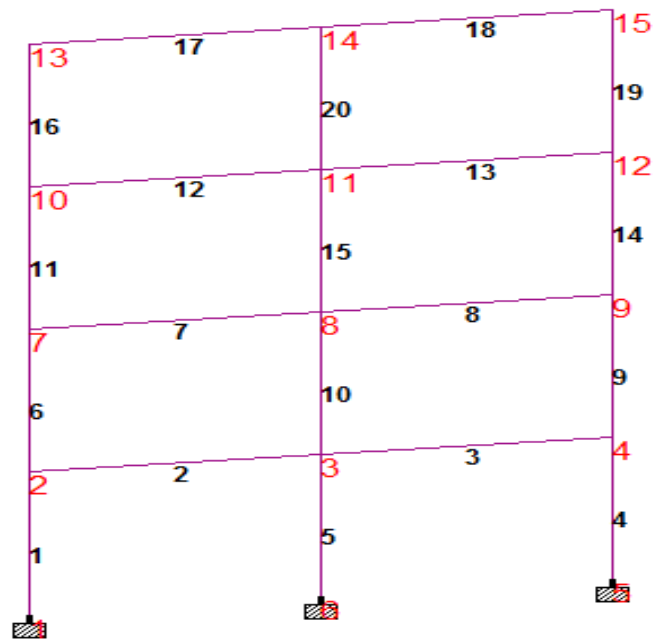


TENSILE STRESS

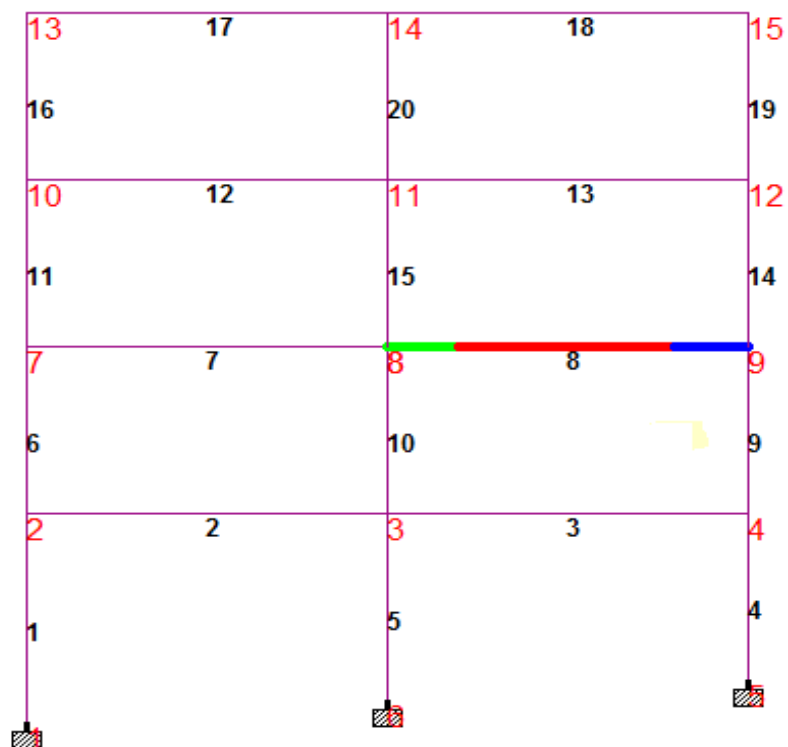
◆ PLANE GROUND ■ SLOPING GROUND



PHASE 2.2(DOUBLE BAY)



PLANE GROUND

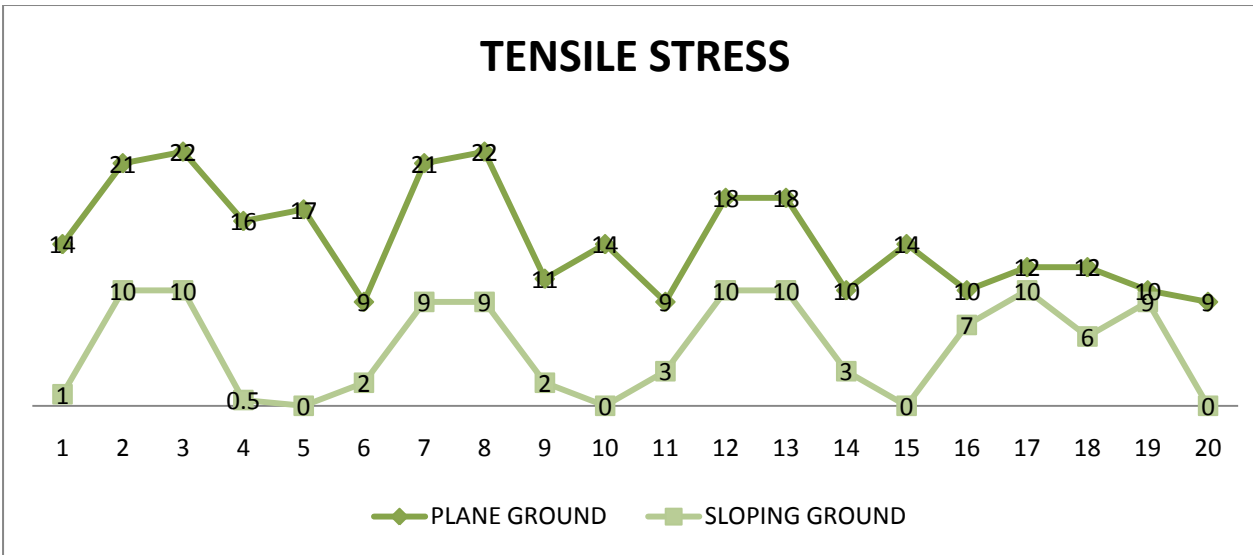
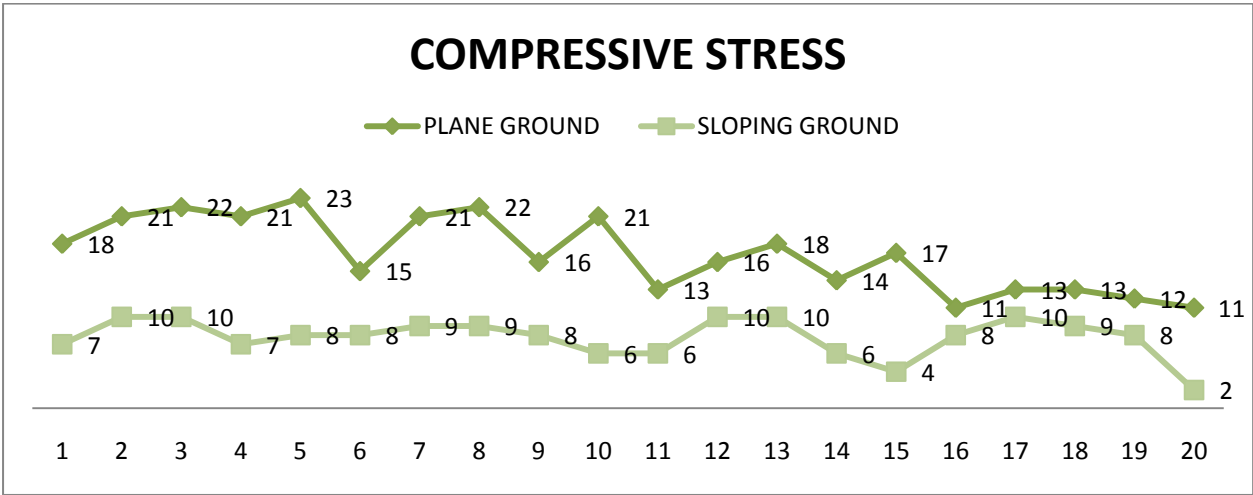
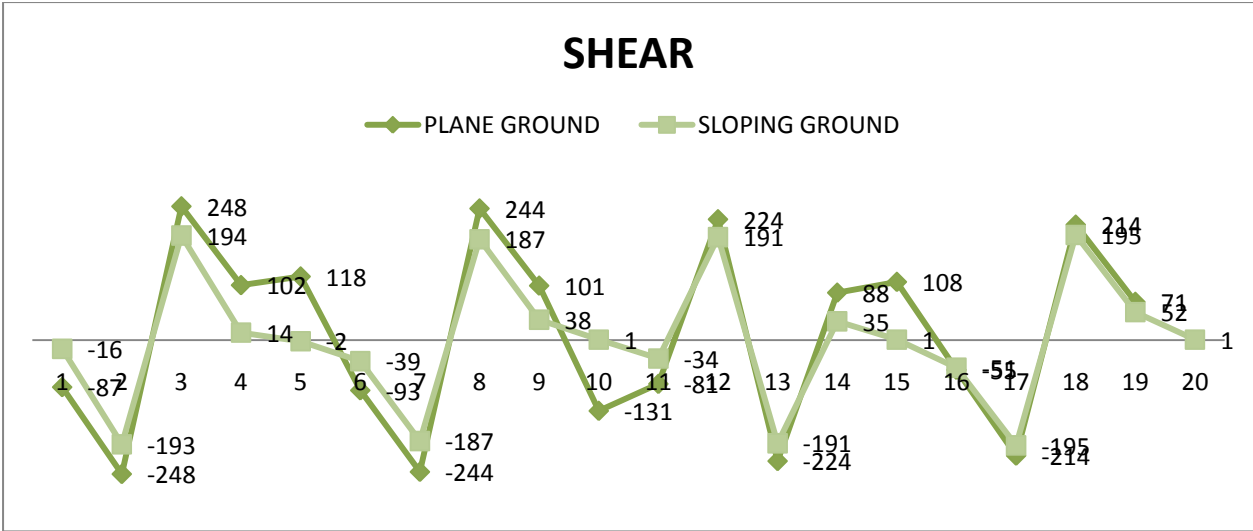


SLOPING GROUND

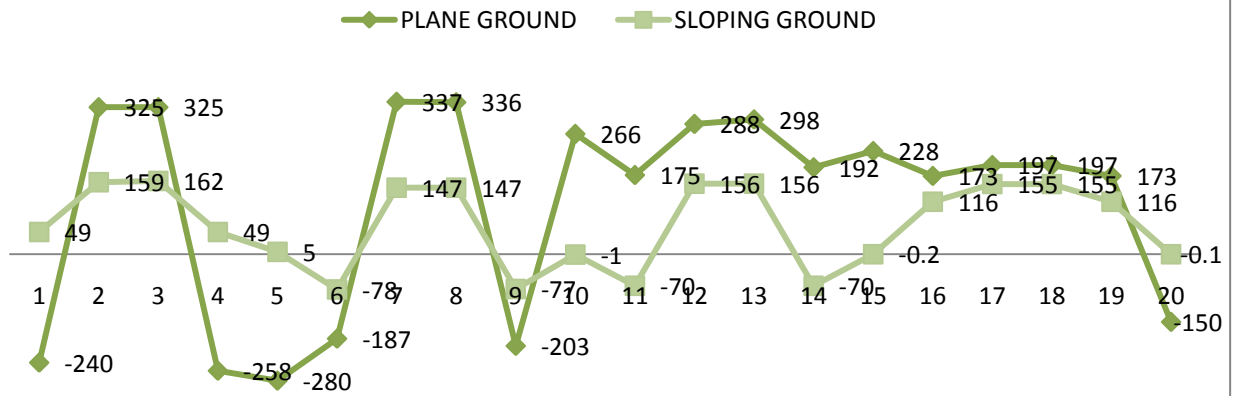
ANALYSIS RESULTS FOR 2 BAY SYSTEMS ON PLANE AND ON A SLOPING GROUND FOR FOUR STORY FRAME

BEAM NO	MAXIMUM AXIAL FORCE		MAXIMUM SHEAR FORCE		MAXIMUM BENDING MOMENT		MAXIMUM TENSILE FORCE		MAXIMUM COMPRESSIVE FORCE	
	kN		kN		kN-m		N/ mm ²		N/ mm ²	
	P*	S*	P	S	P	S	P	S	P	S
1	978	743	-87	-16	-240	49	14	1	18	7
2	-22	-23	-248	-193	325	159	21	10	21	10
3	23	-21	248	194	325	162	22	10	22	10
4	978	738	102	14	-258	49	16	.5	21	7
5	1780	1530	118	-2	-280	5	17	0	23	8
6	718	558	-93	-39	-187	-78	9	2	15	8
7	4	4	-244	-187	337	147	21	9	21	9
8	5	4	244	187	336	147	22	9	22	9
9	718	558	101	38	-203	-77	11	2	16	8
10	1330	1130	-131	1	266	-1	14	0	21	6
11	453	371	-81	-34	175	-70	9	3	13	6
12	-18	-17	224	191	288	156	18	10	16	10
13	-18	-17	-224	-191	298	156	18	10	18	10
14	461	371	88	35	192	-70	10	3	14	6
15	887	757	108	1	228	-.2	14	0	17	4
16	224	181	-55	-51	173	116	10	7	11	8
17	55	53	-214	-195	197	155	12	10	13	10
18	55	53	214	195	197	155	12	6	13	9
19	224	180	71	52	173	116	10	9	12	8
20	456	390	65	1	-150	-.1	9	0	11	2

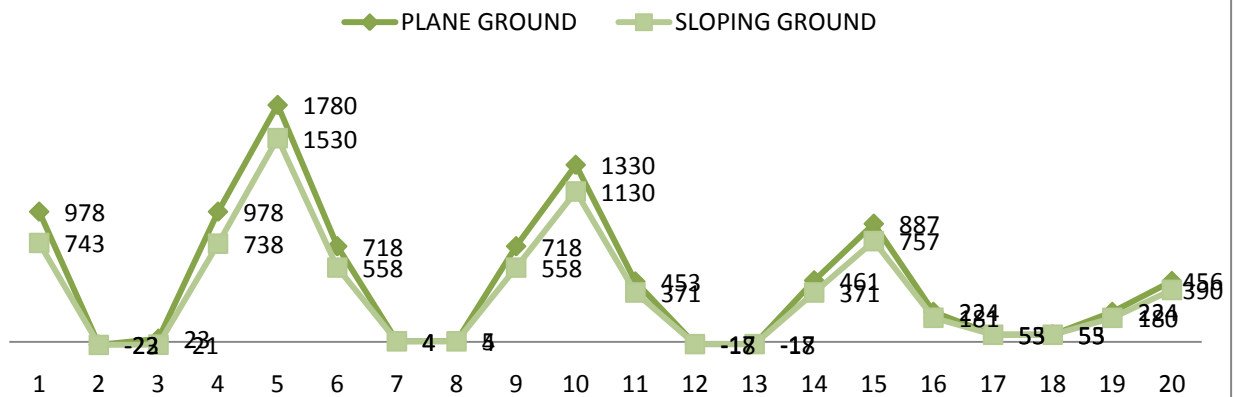
[P= Plane ground, S= Sloping Ground]*



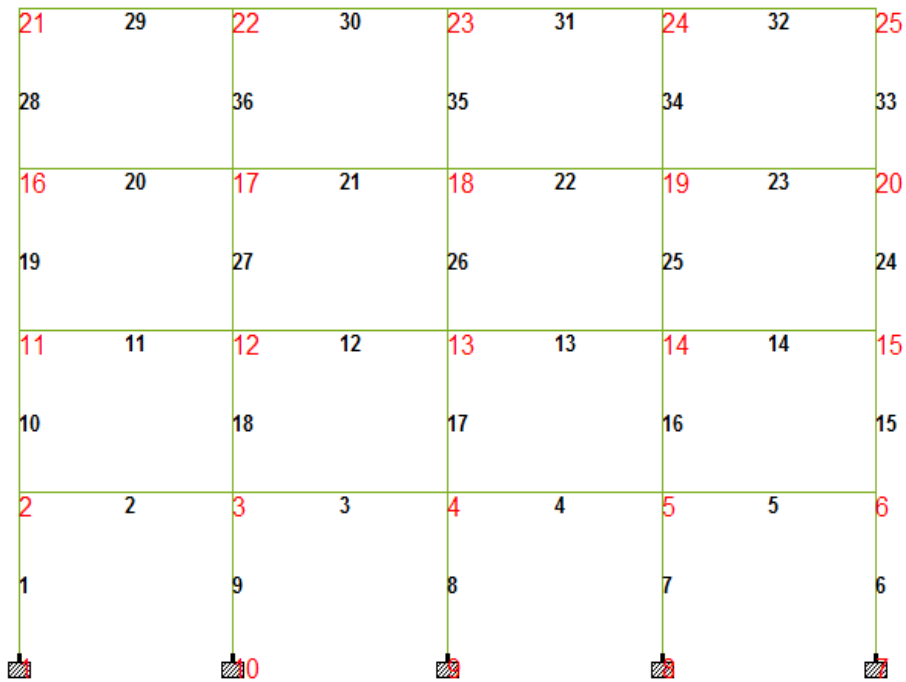
BENDING MOMENT



AXIAL FORCE



PHASE 2.2(4 BAY SYSTEMS)



PLANE GROUND



SLOPING GROUND

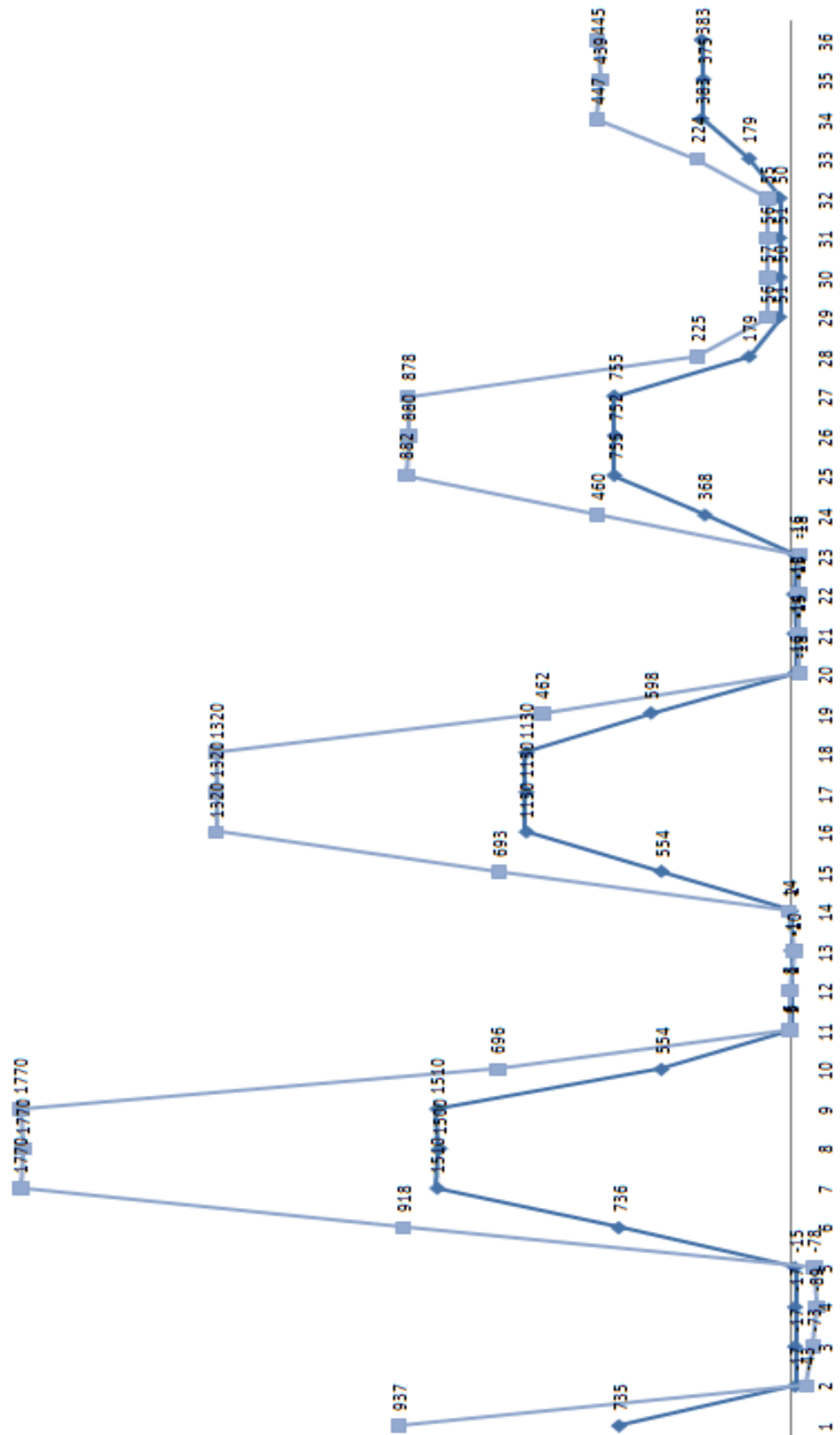
ANALYSIS RESULTS FOR 4 BAY SYSTEMS ON PLANE AND ON A SLOPING GROUND FOR FOUR STORY FRAME

BEAM NO	MAXIMUM AXIAL FORCE		MAXIMUM SHEAR FORCE		MAXIMUM BENDING MOMENT		MAXIMUM TENSILE FORCE		MAXIMUM COMPRESSIVE FORCE	
	kN		kN		kNm		N/ mm ²		N/ mm ²	
	P*	S*	P	S	P	S	P	S	P	S
1	735	937	-20	-36	54	-137	0.1	4	7	12
2	-17	-43	-193	-222	160	264	10	16	10	16
3	-17	-73	-187	-212	157	259	10	16	10	16
4	-17	-89	-187	-216	157	271	10	17	10	17
5	-15	-78	193	238	160	318	10	19	7	18
6	736	918	21	125	54	-297	0.1	16	7	22
7	1510	1770	-0.4	99	-1.2	-252	0	12	8	21
8	1500	1770	-0.1	-72	-0.5	208	0	9	8	17
9	1510	1770	0.4	-55	1.265	174	0	7	8	16
10	554	696	-37	-77	-75	-156	2	6	8	13
11	3	6	-188	-216	150	267	8	16	9	16
12	1	8	-187	-212	157	256	9	15	9	15
13	-1	-10	-187	-210	157	254	9	15	9	15
14	2	14	188	216	150	267	9	16	9	16
15	554	693	37	66	-75	146	2	6	7	12
16	1130	1320	-0.5	84	2	173	0	8	6	15
17	1130	1320	0	83	0	170	0	8	6	14
18	1130	1320	0.5	87	-2	-177	0	8	6	15
19	598	462	-35	-62	-70	134	2	7	6	10
20	-16	-18	189	209	153	231	9	14	9	14
21	-14	-15	-188	-207	158	223	10	13	9	13
22	-12	-15	188	207	158	221	10	13	9	13
23	-16	-18	-189	-207	153	230	9	14	9	14
24	368	460	35	65	-70	141	2	6	6	10
25	755	882	3	70	6	148	0	7	4	12
26	752	880	0	68	0	143	0	7	4	11
27	755	878	3	65	7	-146	0	7	4	11
28	179	225	-50	-60	114	145	6	8	8	10
29	51	56	-195	-213	156	169	9	10	10	10
30	50	57	-187	-205	156	174	9	10	10	11
31	51	56	-187	-205	156	174	9	10	10	11
32	50	55	195	214	156	172	9	10	10	11
33	179	224	51	60	114	144	6	8	8	10
34	383	447	0.5	40	-0.2	91	0	5	2	7
35	375	439	0	40	0	92	0	5	2	7
36	383	445	0.5	-39	2	-90	0	5	2	7

[P= Plane ground, S= Sloping Ground]*

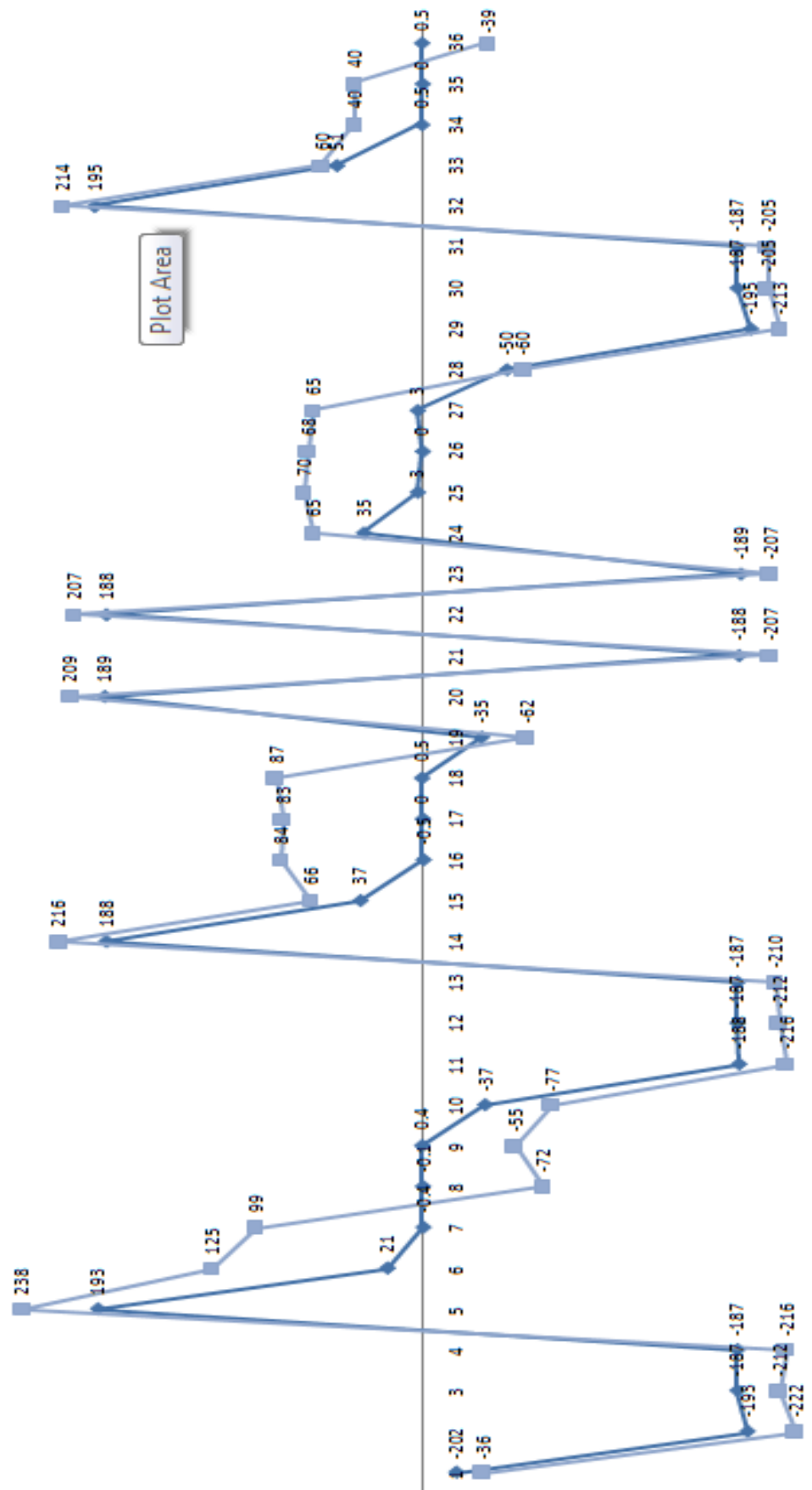
AXIAL FORCE

◆ PLANE GROUND ■ SLOPING GROUND



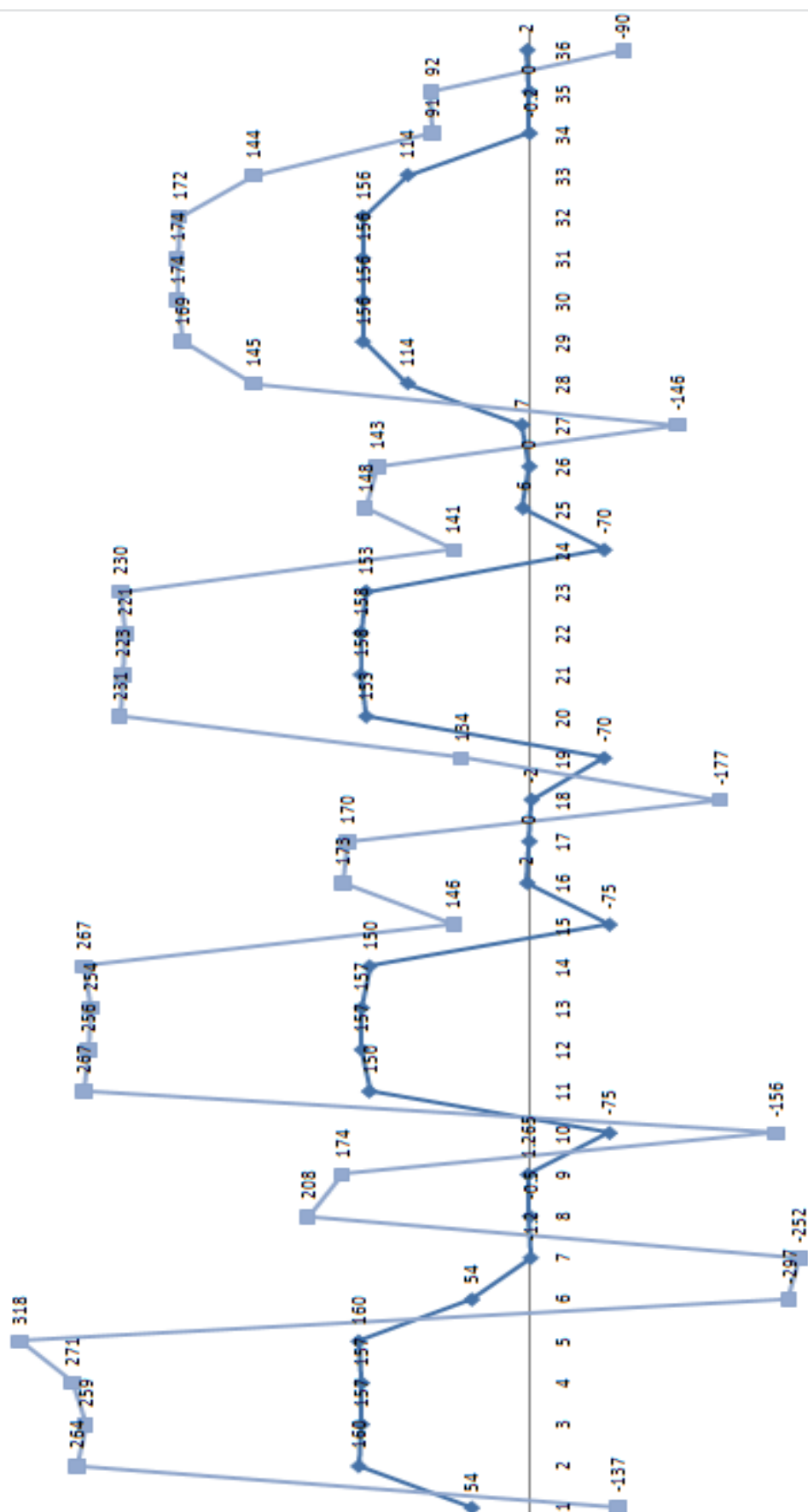
SHEAR

◆ PLANE GROUND ■ SLOPING GROUND

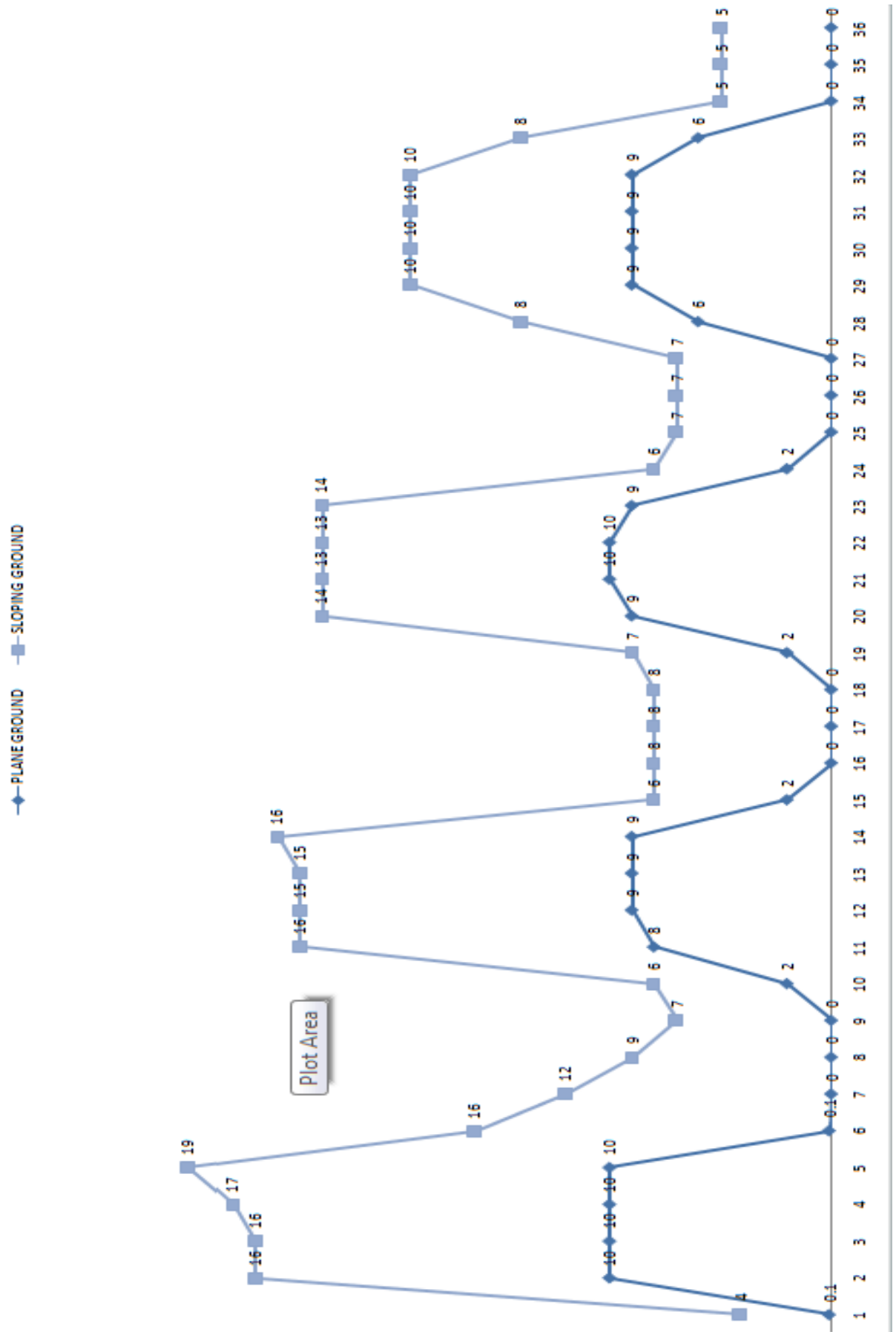


BENDING MOMENT

—◆— PLANE GROUND —■— SLOPING GROUND

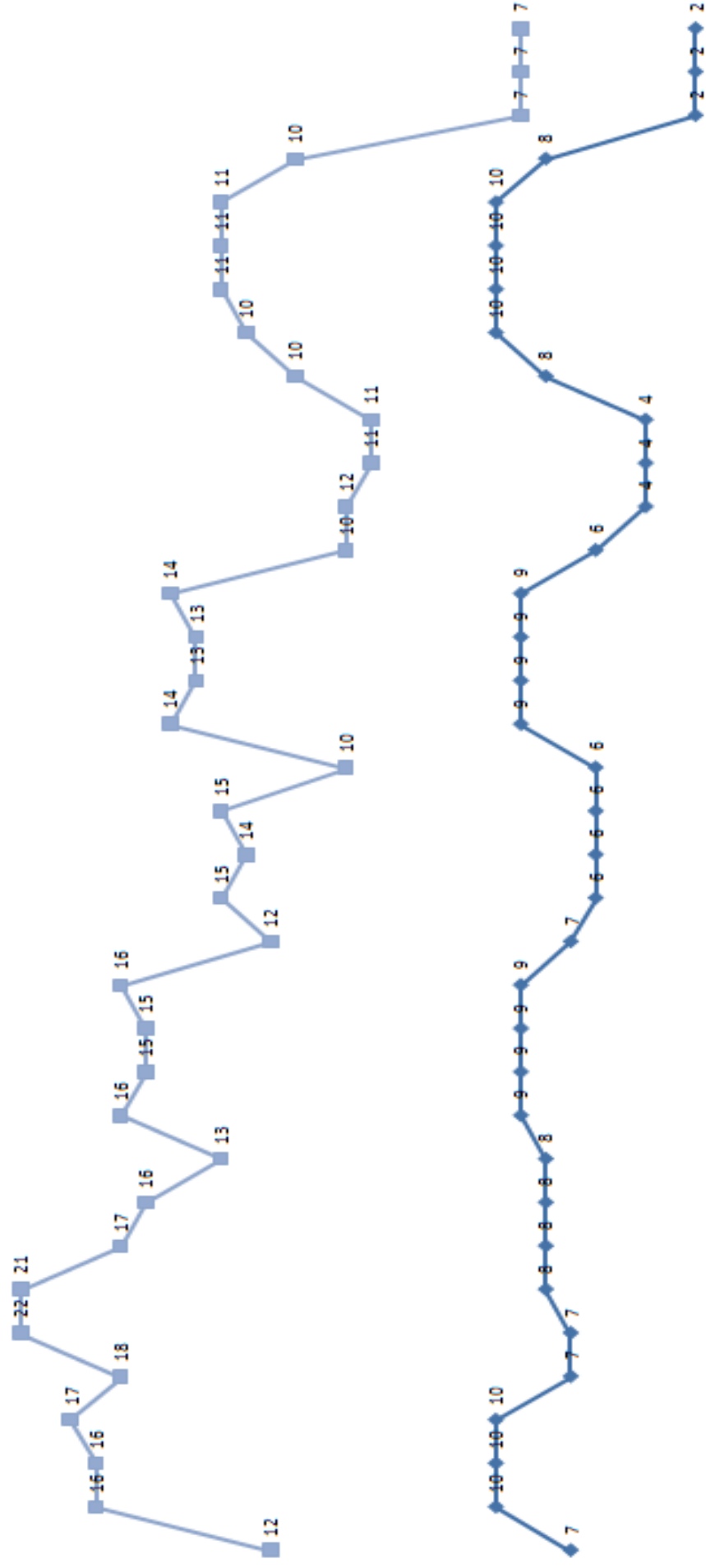


TENSILE STRESS



COMPRESSIVE STRESS

—◆— PLANE GROUND —■— SLOPING GROUND



PHASE 2.3(DOUBLE BAY)



PLANE GROUND



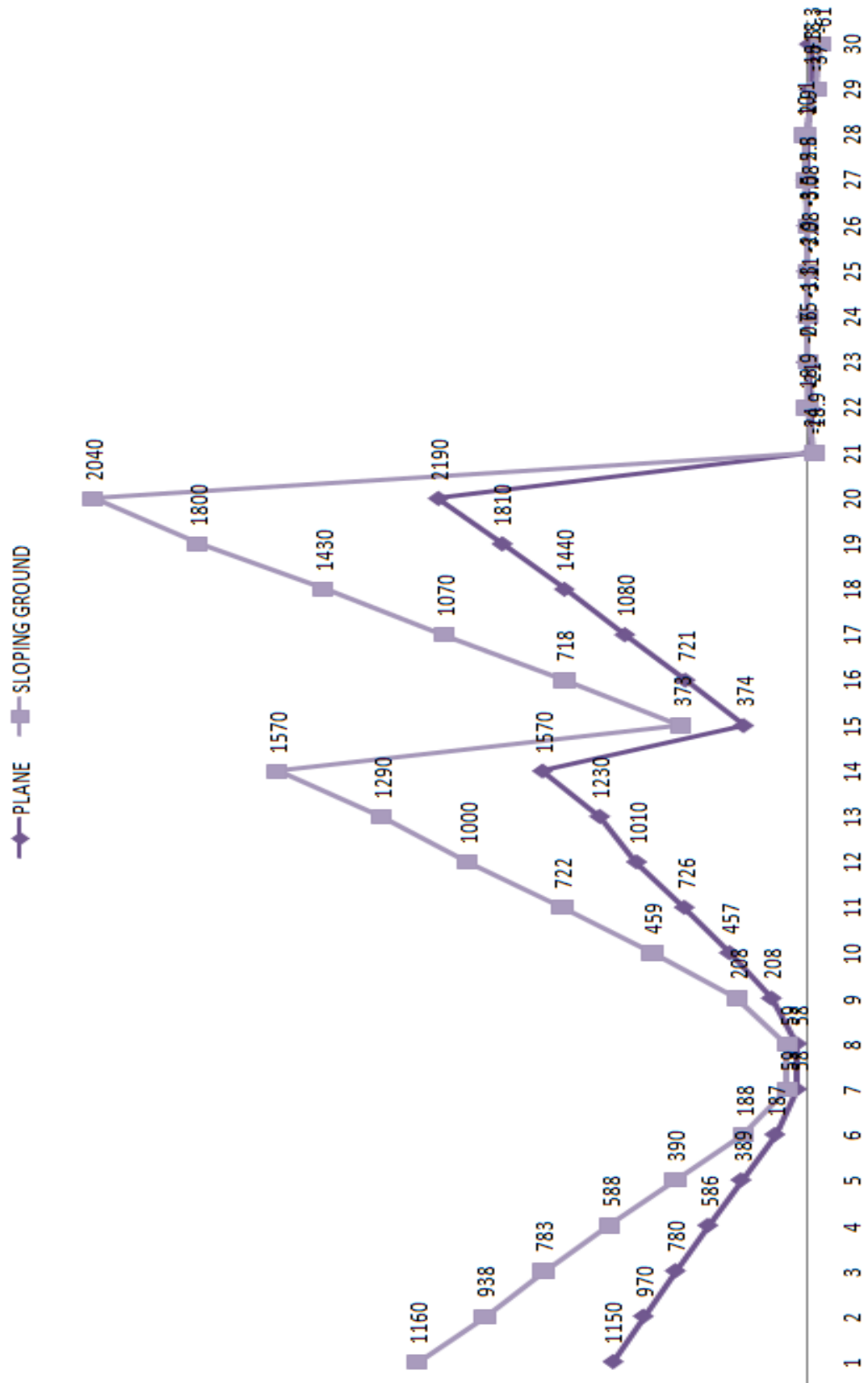
SLOPING GROUND

ANALYSIS RESULTS FOR 2 BAY SYSTEMS ON PLANE AND ON A SLOPING GROUND FOR SIX STORY FRAME

BEAM NO	MAXIMUM AXIAL FORCE		MAXIMUM SHEAR FORCE		MAXIMUM BENDING MOMENT		MAXIMUM TENSILE STRESS		MAXIMUM COMPRESSIVE STRESS	
	kN		kN		kN-m		N/ mm ²		N/ mm ²	
	P*	S*	P	S	P	S	P	S	P	S
1	1150	1160	-103	-70	-235	-229	14.6	11.6	19.9	17.9
2	970	938	-96	-99	-199	-207	9.3	9.8	16.6	17.1
3	780	783	-93	-88	192	188	9.2	7.6	15.1	14.9
4	586	588	-88	-86	178	182	9.3	9	13.8	13.5
5	389	390	-73	-71	161	157	8.2	7.8	11.3	11.1
6	187	188	-58	-62	155	153	8.5	8.3	10.1	9.9
7	58	59	-187	-186	164	160	9.5	9.3	10.7	9.87
8	58	59	-187	-188	175	174	10.3	10.2	11.4	10.1
9	208	208	72	66	175	174	9.6	9.5	13	11.4
10	457	459	83	81	185	180	9.1	8.9	16.1	11.7
11	726	722	101	98	214	210	9.6	9.4	18	15.8
12	1010	1000	109	107	223	220	9.7	9.4	20.1	17.8
13	1230	1290	112	103	-233	-203	11	9.5	24.8	18.6
14	1570	1570	112	146	-288	-361	17.7	22.5	8.6	19.4
15	374	373	95	100	122	115	6.8	6.4	14.8	8.2
16	721	718	126	91	264	195	11.4	10.8	19.7	14.3
17	1080	1070	126	121	261	252	14.5	14	25	19.1
18	1440	1430	143	138	290	225	15.9	13.9	27	22.3
19	1810	1800	151	151	311	281	17	16.8	23	25.5
20	2190	2040	132	122	324	322	16.8	13	19.9	27.2
21	-24	-18.9	258	202	348	256	22.4	13.4	23.5	12.7
22	-21	18.9	-249	-224	392	284	23	17.2	23.2	16.8
23	-0.6	-2.75	-260	-224	386	281	25	18.7	25.8	18.7
24	-1.3	-3.11	-264	-226	431	388	25	21	21.4	21.3
25	-2.9	-3.08	-249	-226	357	350	21.4	21.6	21.4	20.9
26	-3.5	-3.08	-260	-245	382	376	24.8	24.3	24.7	24.2
27	2.8	5.5	-230	-258	319	368	19.1	22.8	18.2	22.19
28	2.9	10.1	-247	-258	363	387	21.8	25.4	21.8	25.3
29	-18.3	-37	-201	-265	261	367	15.8	21.9	15.6	22
30	-18.3	-61	-225	-257	288	391	17.4	25.2	17.2	25.5

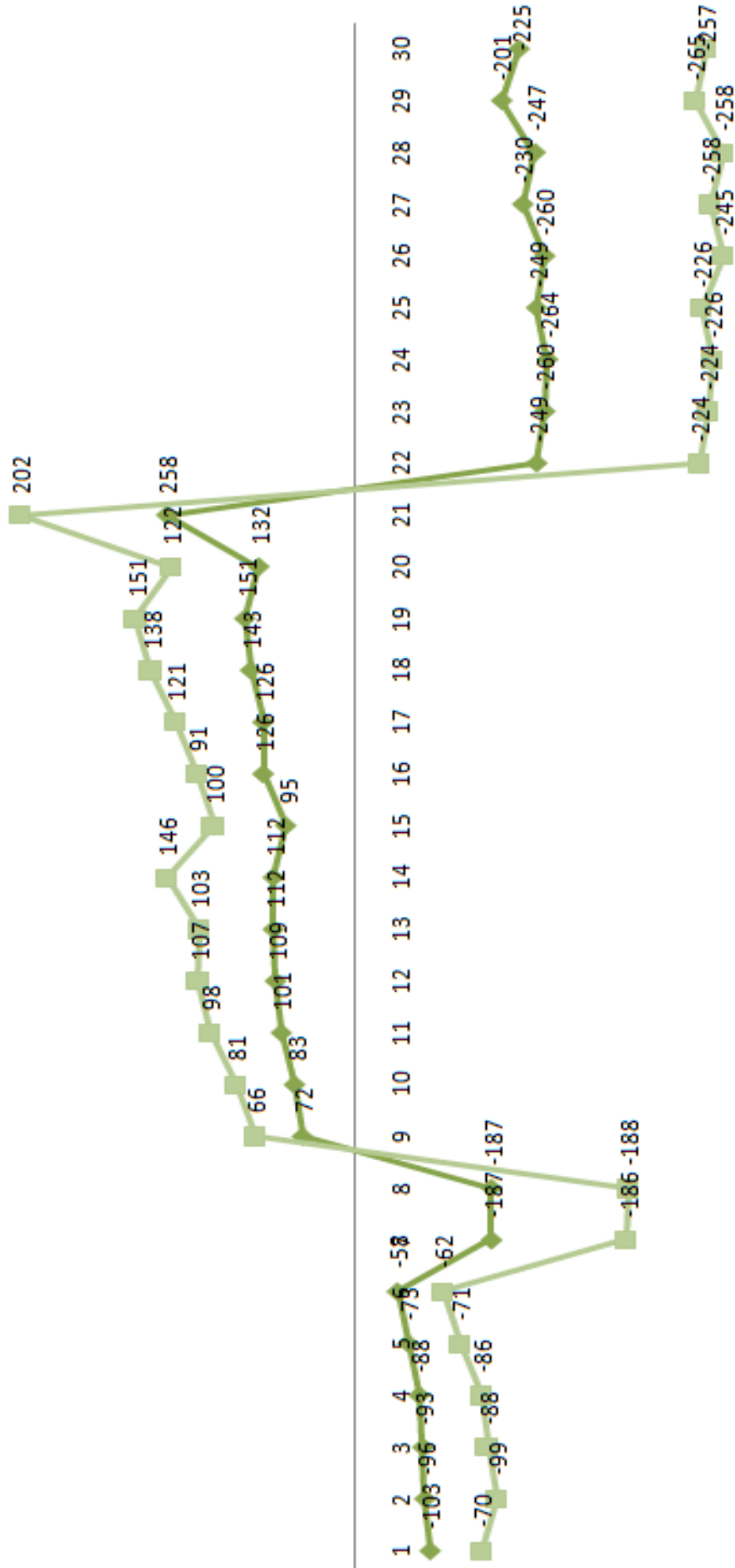
[P= Plane ground, S= Sloping Ground]*

MAXIMUM AXIAL FORCE

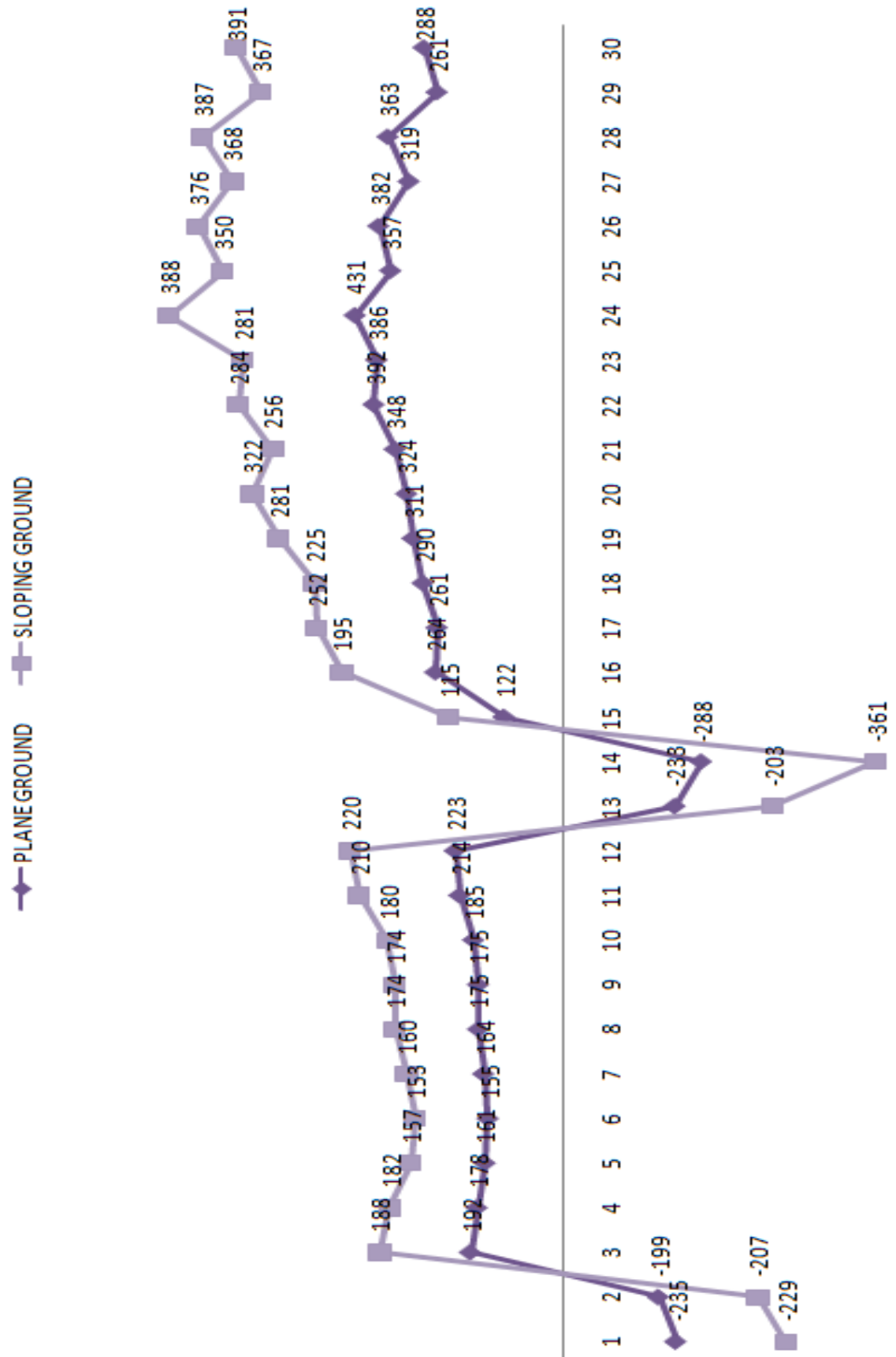


MAXIMUM SHEAR FORCE

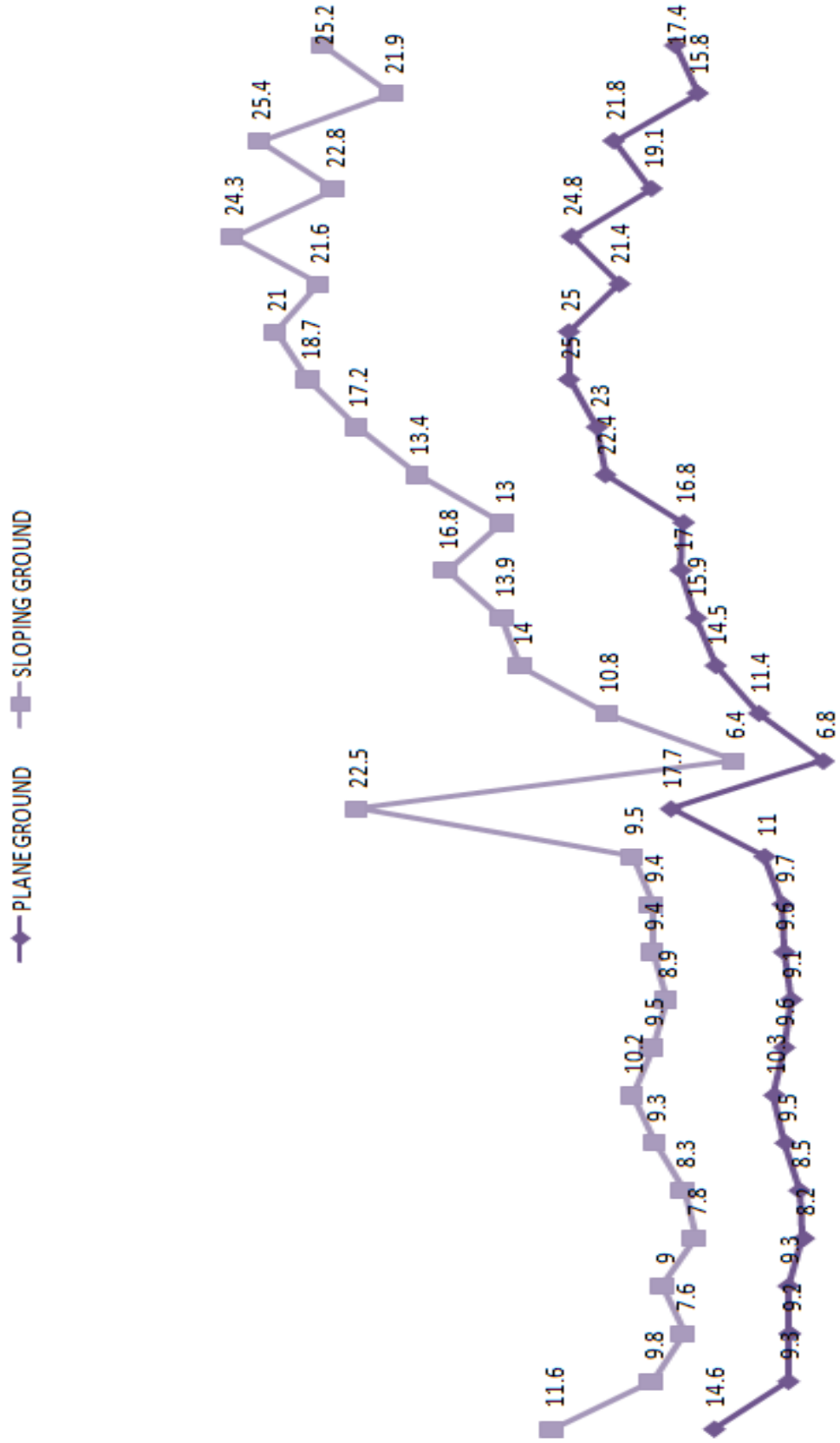
◆ PLANE GROUND
 ■ SLOPING GROUND



MAXIMUM BENDING MOMENT

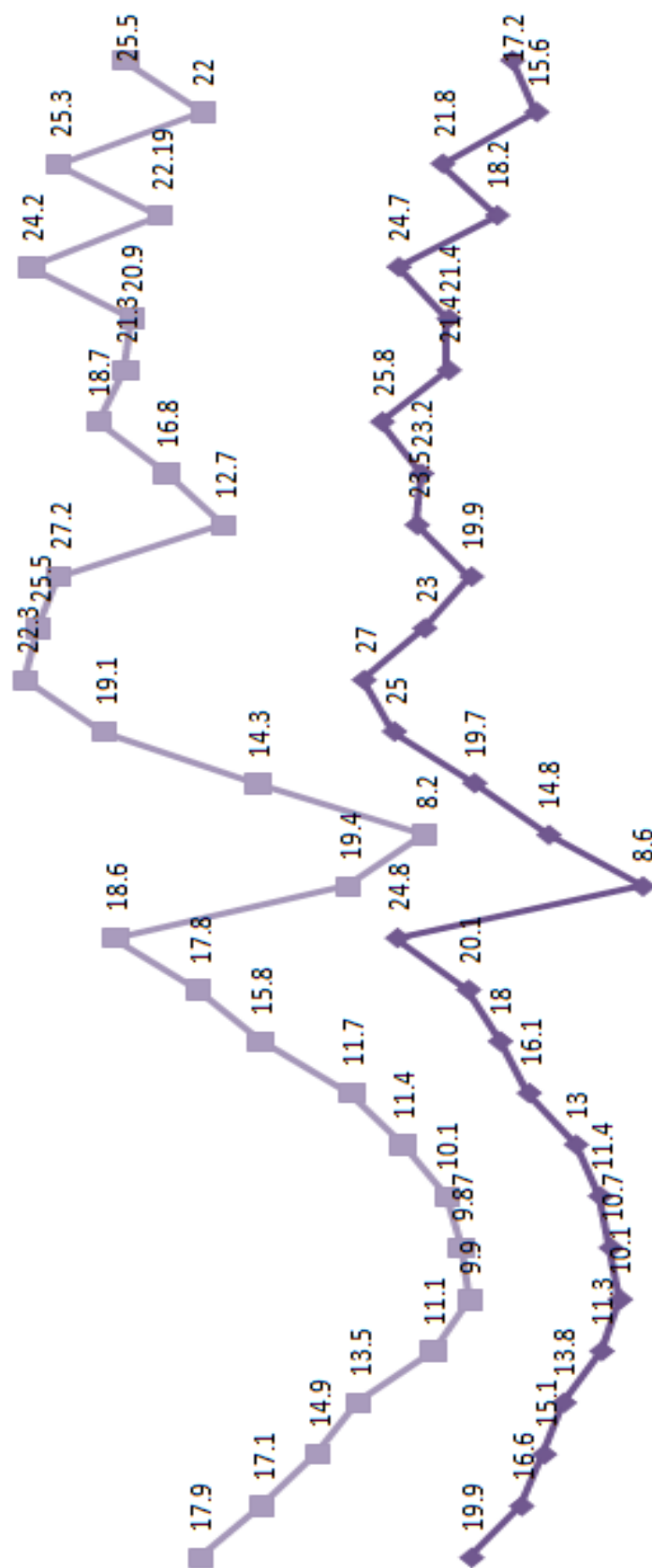


MAXIMUM TENSILE STRESS

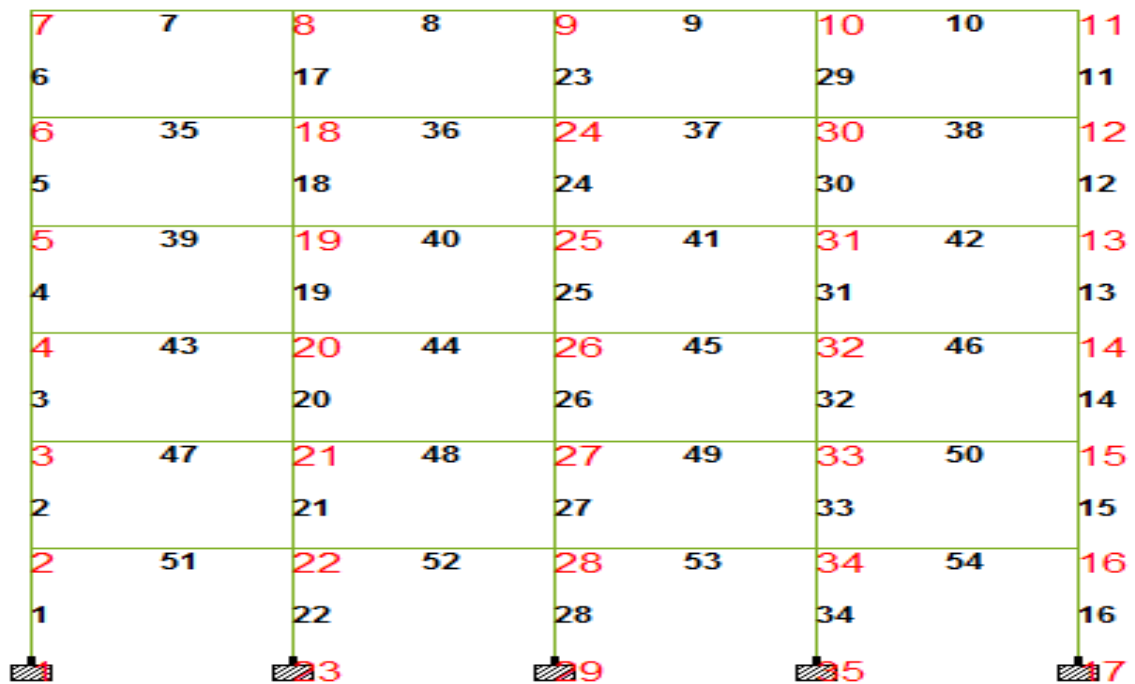


MAXIMUM COMPRESSIVE STRESS

◆ PLANE GROUND ■ SLOPING GROUND



PHASE 2.3(FOUR BAY)



PLANE GROUND



SLOPING GROUND

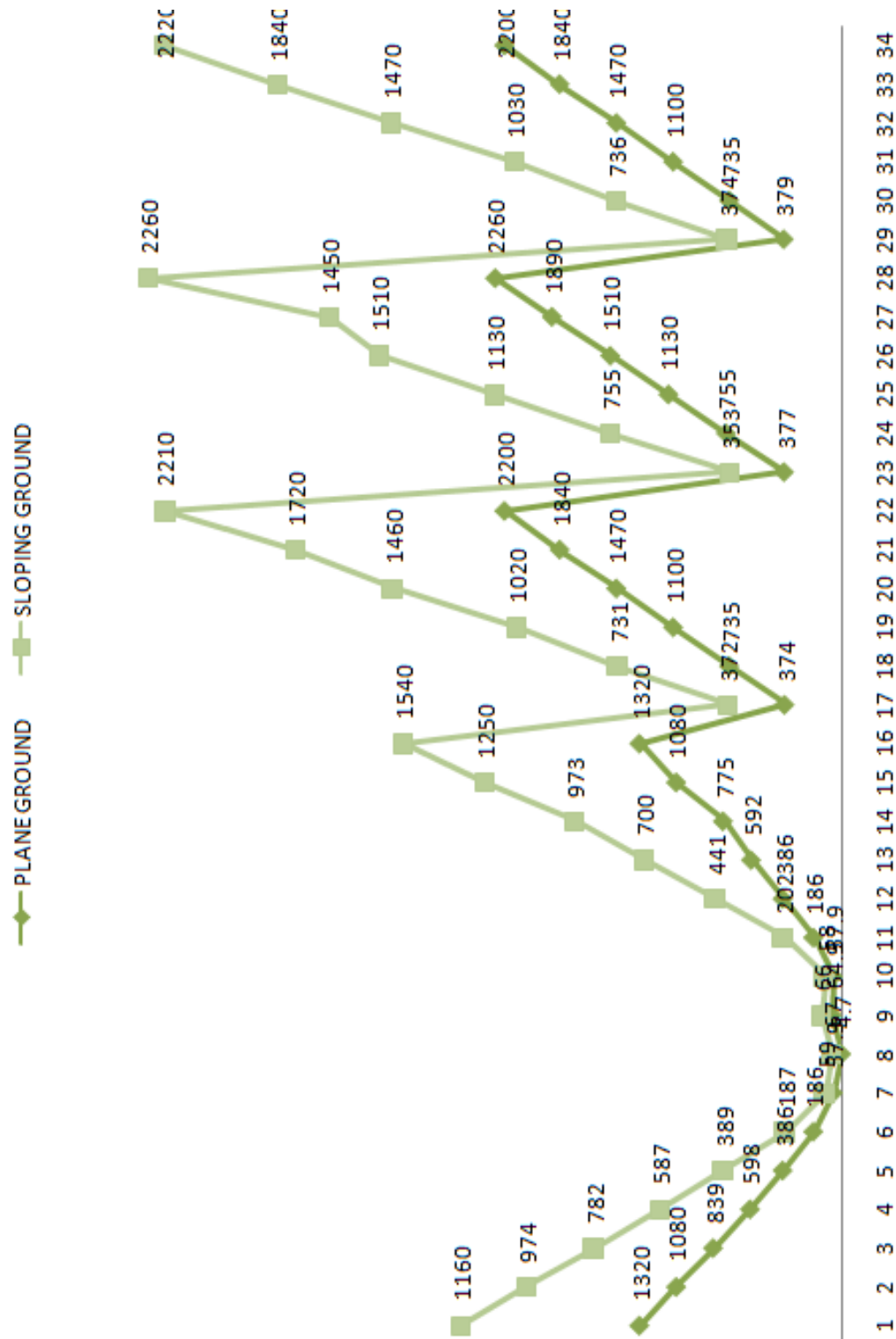
**ANALYSIS RESULTS FOR 4 BAY SYSTEMS ON PLANE AND ON A SLOPING
GROUND FOR SIX STORY FRAME**

BEAM NO	MAXIMUM AXIAL FORCE		MAXIMUM SHEAR FORCE		MAXIMUM BENDING MOMENT		MAXIMUM TENSILE STRESS		MAXIMUM COMPRESSIVE STRESS	
	kN		kN		kN-m		N/ mm ²		N/ mm ²	
	P*	S*	P	S	P	S	P	S	P	S
1	1320	1160	-107.6	-53	-282	-203	16.2	9.8	21.5	15.8
2	1080	974	-95.6	-97	-196	-201	8.6	9.3	17.9	16.4
3	839	782	-93	-82	190	179	8.5	7.2	15.6	14.2
4	598	587	-87	-18	-164	-171	8.7	8.3	14	12.7
5	386	389	-87	-67	161	148	8	7.3	11.5	10.4
6	186	187	-74	-59	156	142	8.5	7.8	10.22	9.3
7	57.9	59	-64.1	-187	156	151	9.1	8.8	9.58	9.3
8	4.7	67	-188.5	-189	174	183	10.2	10.6	10.76	11.3
9	64.9	66	-188.6	188	174	180	10.2	10.5	10.76	11.1
10	57.9	58	188.6	188	156	162	9.1	9.5	9.58	9.9
11	186	202	64.1	67	156	149	8.5	8.8	10.22	10.6
12	386	441	76.2	75.8	176	168	8.6	8.2	14.4	11.9
13	592	700	92.65	91.7	184	196	9.2	8.6	14.4	14.8
14	775	973	100.76	102	206	-208	8.7	8.6	16.1	16.9
15	1080	1250	104.7	90.3	-214	199	8.6	8.6	17.7	17.6
16	1320	1540	112.6	187	-289	-455	16.2	27.9	21.5	34.6
17	374	372	-51	-45	-119	-106	6.4	5.9	7.8	6.9
18	735	731	-93	84	-200	-182	10.6	10.1	13.3	12.5
19	1100	1020	-120	-111	-222	-232	13	12.7	17.6	17.2
20	1470	1460	-135	-127	-275	-260	13.9	14.1	20.4	20.5
21	1840	1720	-142	-141	289	289	14.1	15.6	22.9	24.4
22	2200	2210	-131	-85	-314	269	15	14.2	25.6	24.5
23	377	353	-52	48	123	113	6.7	6.2	8.5	7.6
24	755	755	91.7	84	197	-181	10.4	10.1	14.1	13.7
25	1130	1130	118.6	111	-246	231	12.7	12.5	18.1	16.7
26	1510	1510	133	126	270	257	13.4	13.7	20.7	19.3
27	1890	1450	-138	-136	-281	-279	13.4	14.5	22.5	23.7
28	2260	2260	-129	-111	-311	322	14.6	17.1	23.9	27.7
29	379	374	53	49	122	114	6.4	5.8	8.5	8.2
30	735	736	95.2	88	204	191	10.6	12.4	14.5	14.2

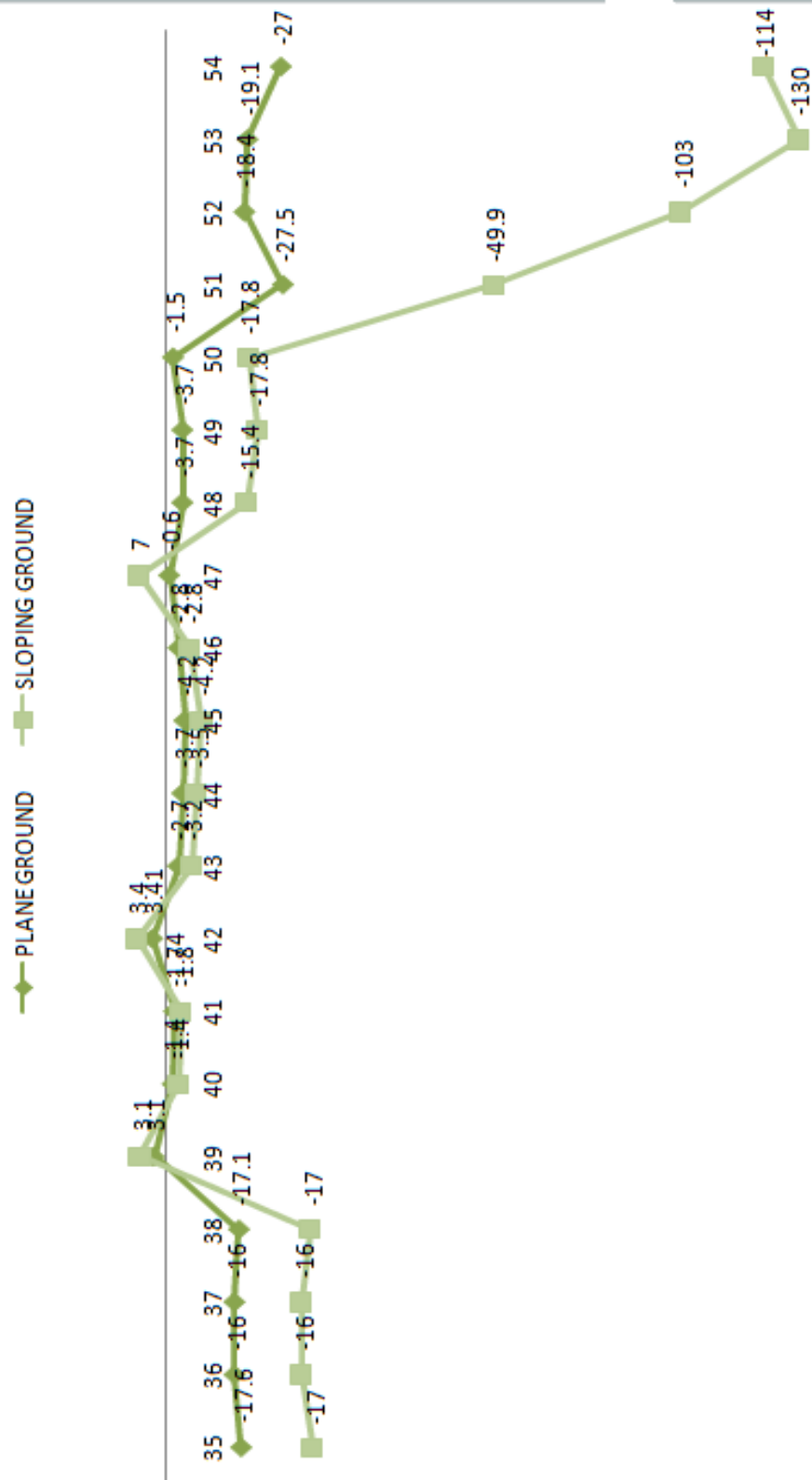
BEAM NO	MAXIMUM AXIAL FORCE		MAXIMUM SHEAR FORCE		MAXIMUM BENDING MOMENT		MAXIMUM TENSILE STRESS		MAXIMUM COMPRESSIVE STRESS	
	kN		kN		kN-m		N/ mm ²		N/ mm ²	
	P	S	P	S	P	S	P	S	P	S
31	1100	1030	122	115	253	239	13	13.75	8.5	18.4
32	1470	1470	137	130	278	265	13.9	14.4	14.5	21.4
33	1840	1840	142	138	-290	278	14.1	21	18.5	23.5
34	2200	2220	131	-154	-314	398	14.6	21.6	21	31.9
35	-17.6	-17	204	-191	261	239	15.7	14.4	22.8	14.3
36	-16	-16	-197	-208	237	243	14.4	14.7	25.3	4.5
37	-16	-16	197	-205	240	238	14.4	14.3	15.6	14.2
38	-17.1	-17	-204	-216	261	266	15.7	16	19.1	15.9
39	3.1	3.1	226	-218	291	293	19.4	17.6	18.1	17.6
40	-1.4	-1.4	-217	-227	291	302	18.1	17.5	17.9	18.1
41	-1.74	-1.8	217	-225	299	299	17.9	18	19.9	17.9
42	3.41	3.4	-226	-236	331	299	19.14	19.8	19.8	19.8
43	-2.7	-3.2	240	-237	355	330	21.3	19.8	21.3	19.8
44	-3.7	-3.5	-230	-240	343	344	20.6	20.6	20.5	20.6
45	-4.2	-4.2	230	-238	340	340	20.4	20.4	20.4	20.4
46	-2.8	-2.8	240	-250	355	379	22.8	22.7	22.7	22.7
47	-0.6	7	245	-250	369	361	21.7	21.6	22.1	21.6
48	-3.7	-15.4	-235	-248	336	368	21.6	22.18	21.6	22
49	-3.7	-17.8	235	-246	358	365	21.5	22	21.5	21.8
50	-1.5	-17.8	-245	-243	400	368	24	21	24	23.8
51	-27.5	-49.9	-242	-251	343	352	21.4	24	21.1	21.2
52	-18.4	-103	227	-241	336	355	20.2	20.9	20.1	19.4
53	-19.1	-130	-227	-247	335	370	20.1	21.6	20.1	22.7
54	-27	-114	242	-267	375	455	22.4	26.8	22.5	27.8

[P= Plane ground, S= Sloping Ground]*

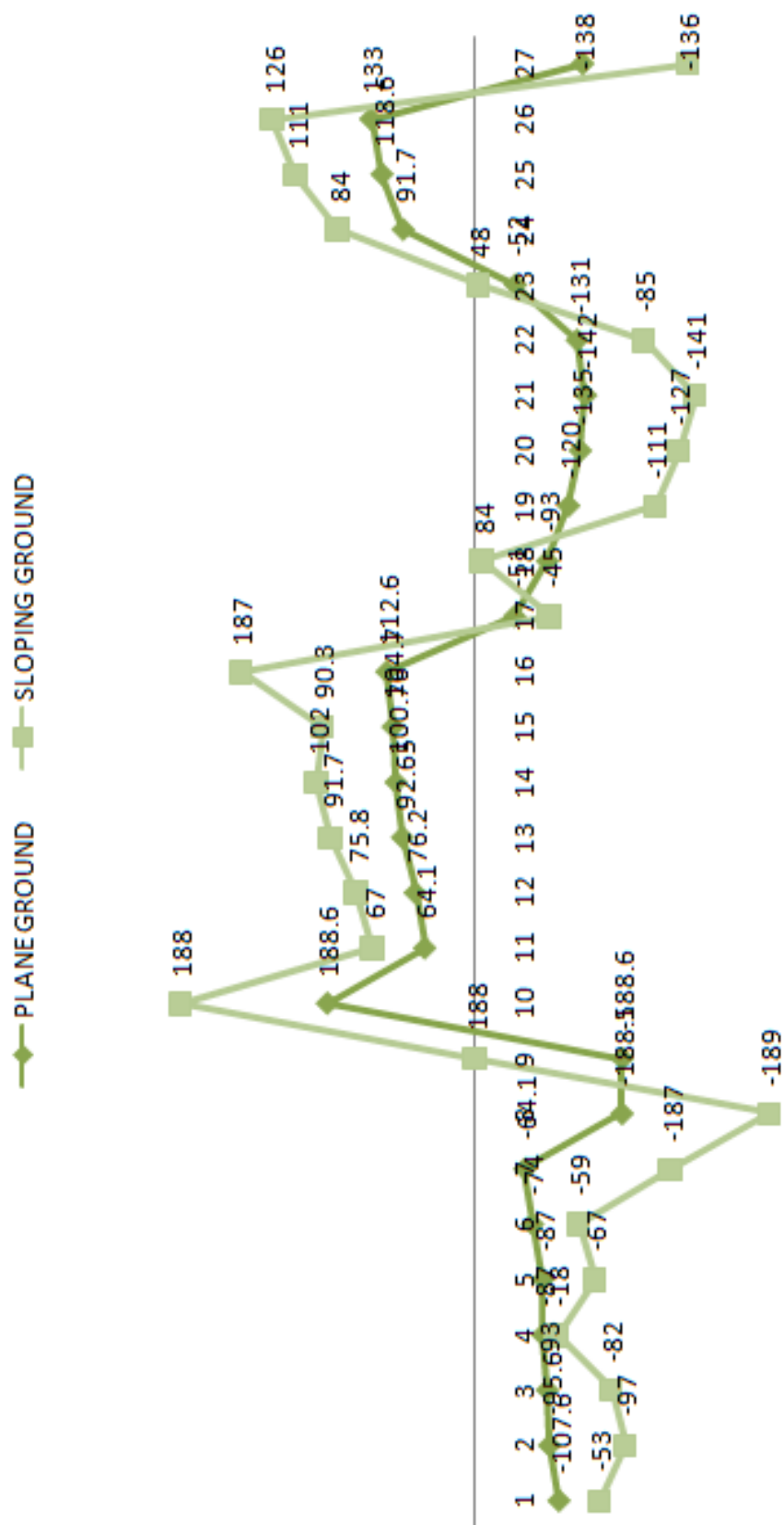
MAXIMUM AXIAL FORCE



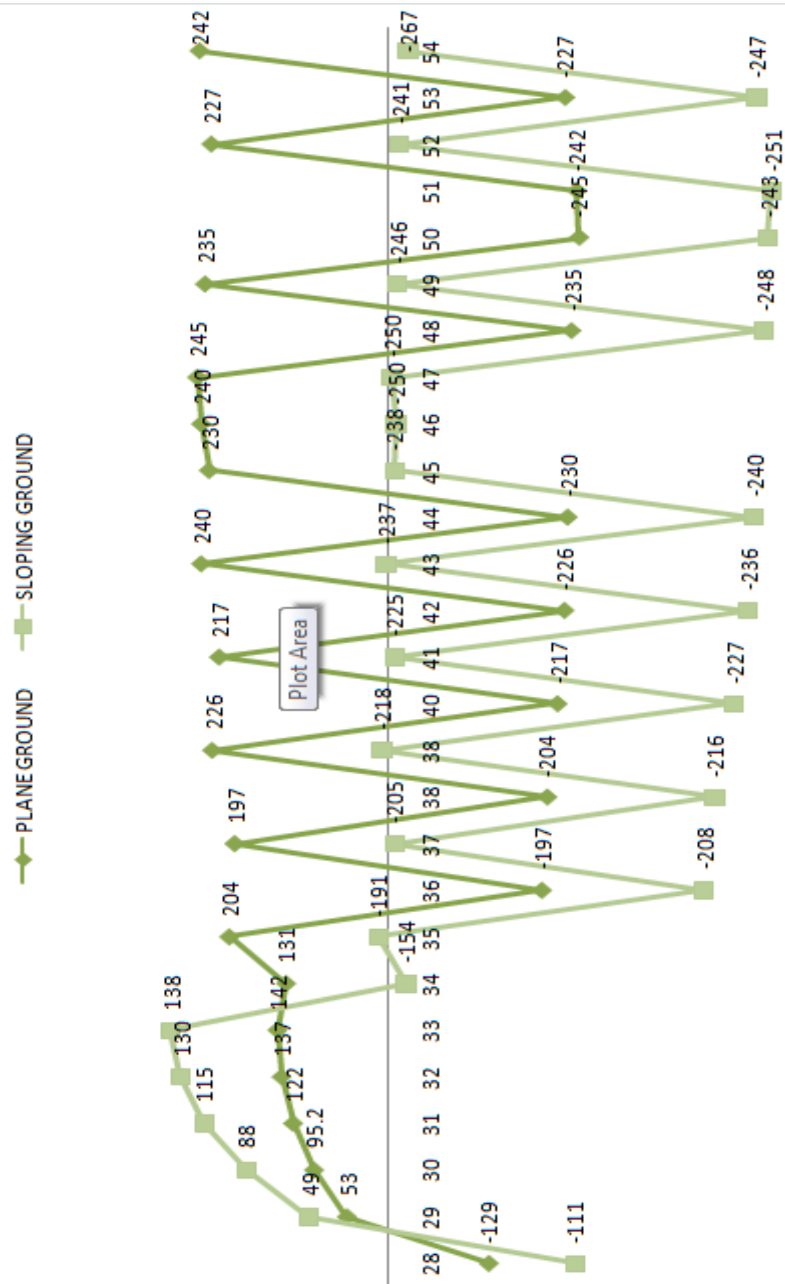
MAXIMUM AXIAL FORCE



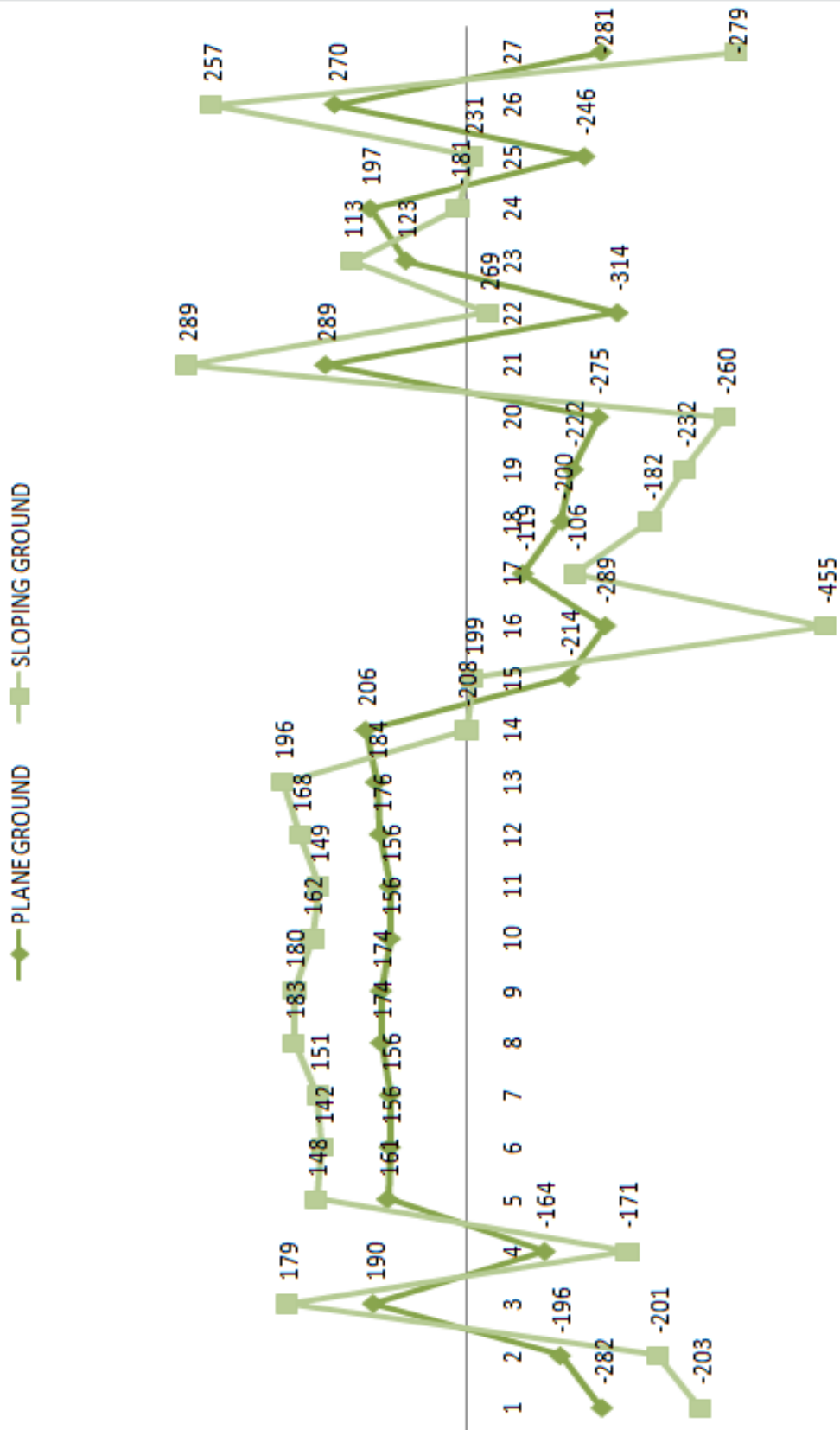
MAXIMUM SHEAR FORCE



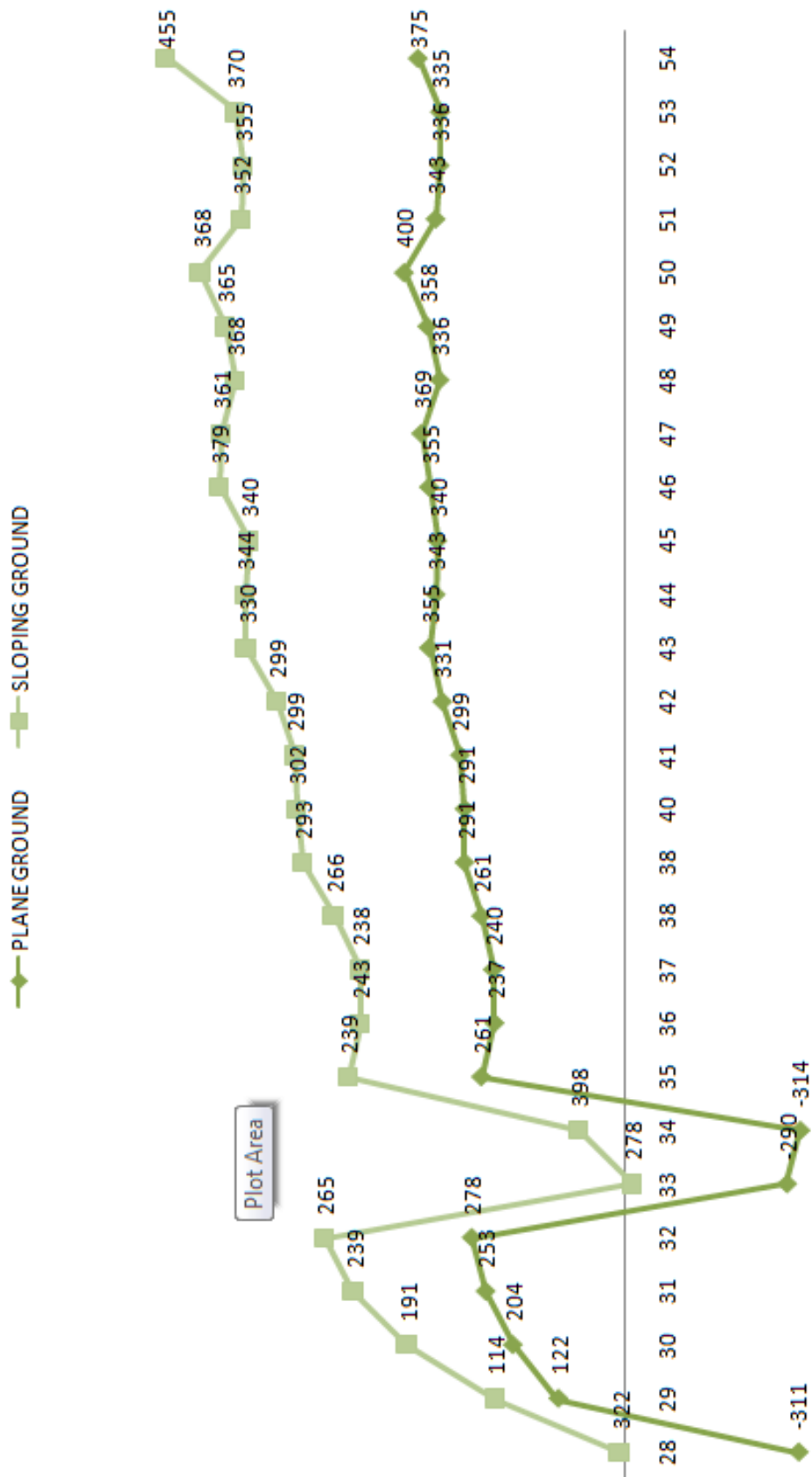
MAXIMUM SHEAR



MAXIMUM BENDING MOMENT

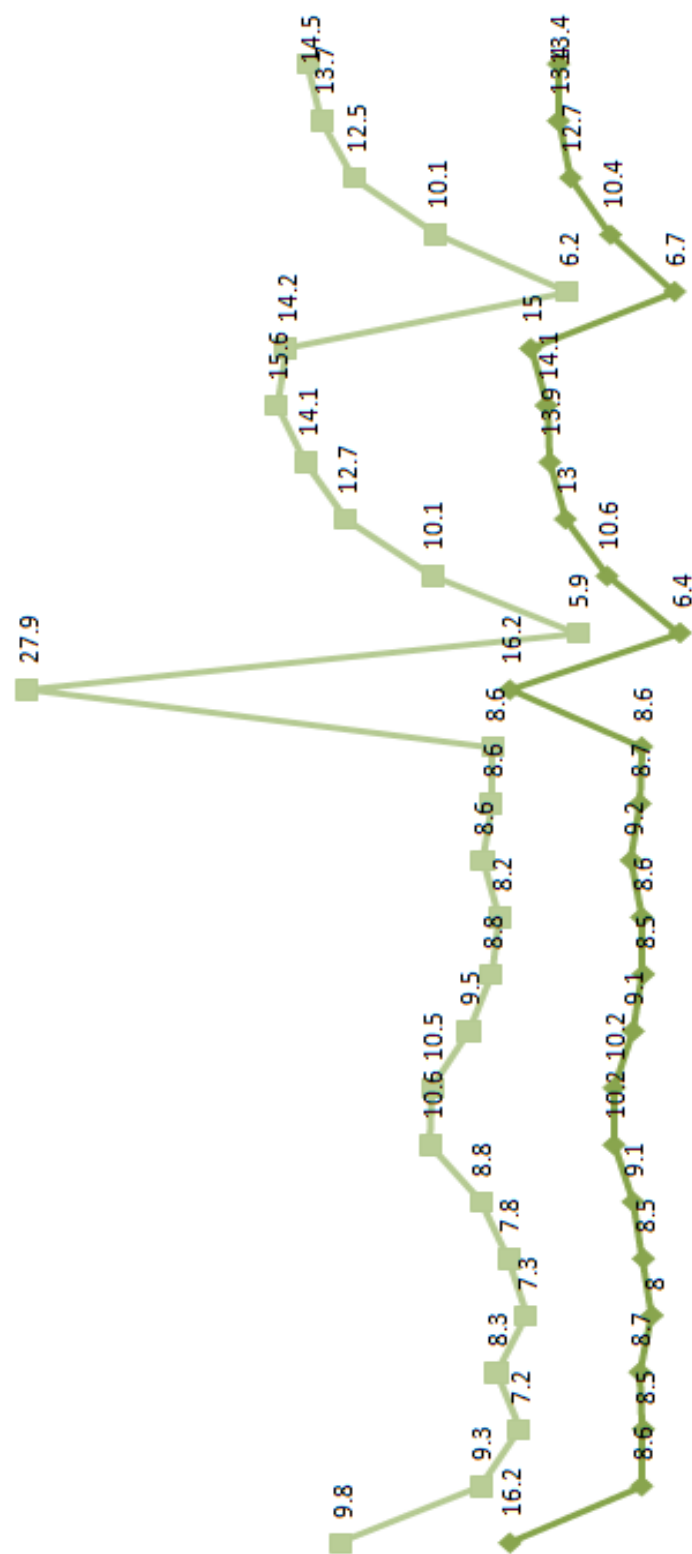


MAXIMUM BENDING MOMENT

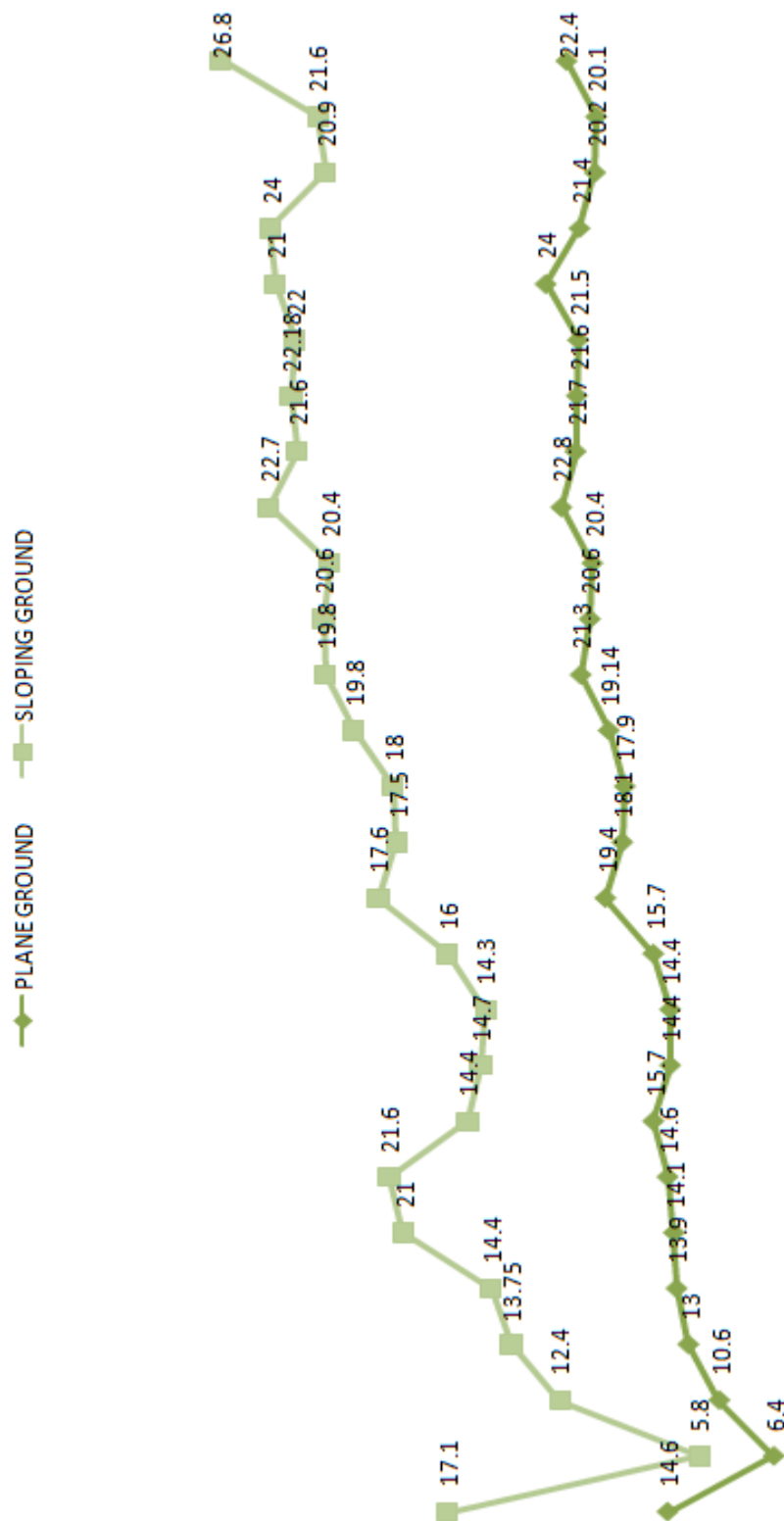


MAXIMUM TENSILE STRESS

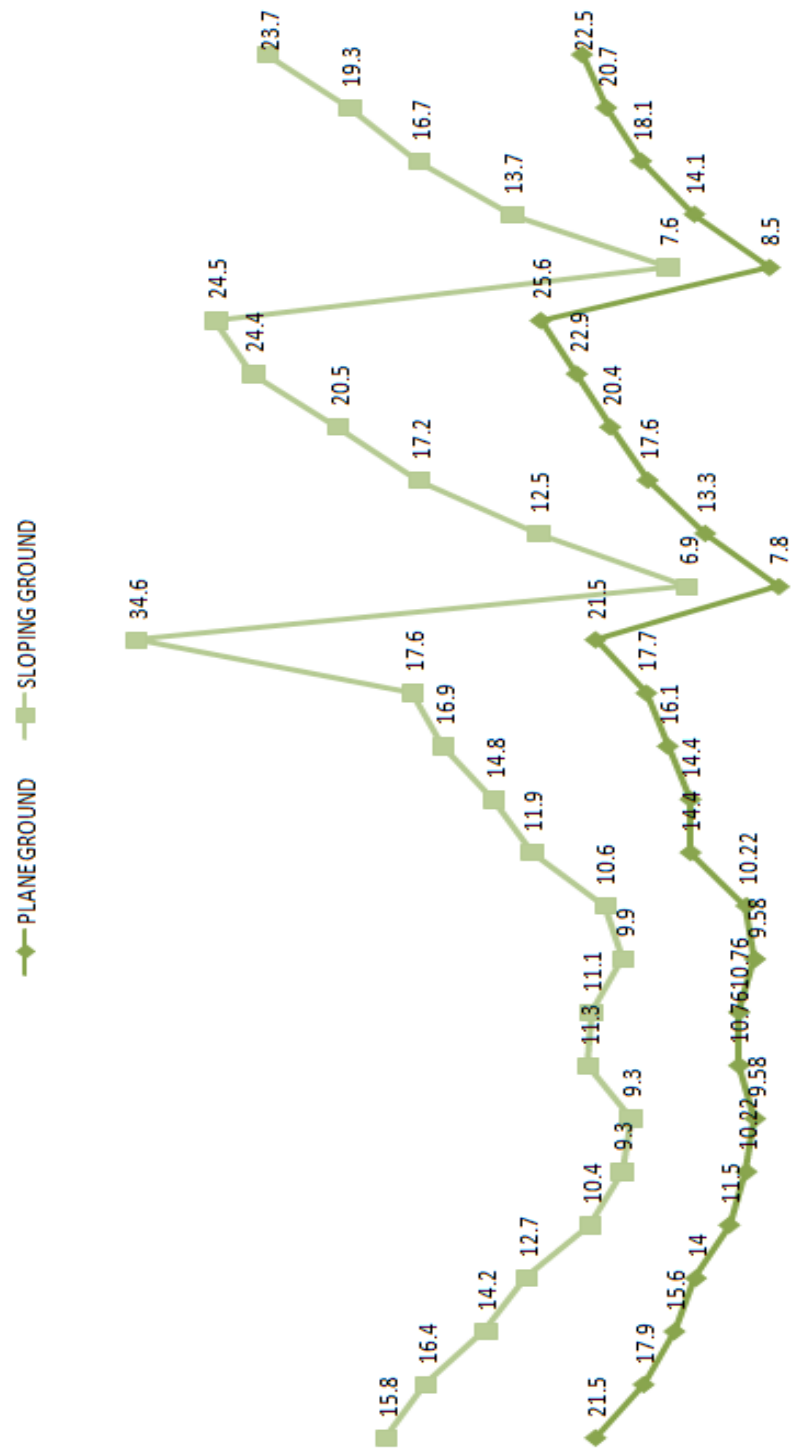
◆ PLANE GROUND ■ SLOPING GROUND



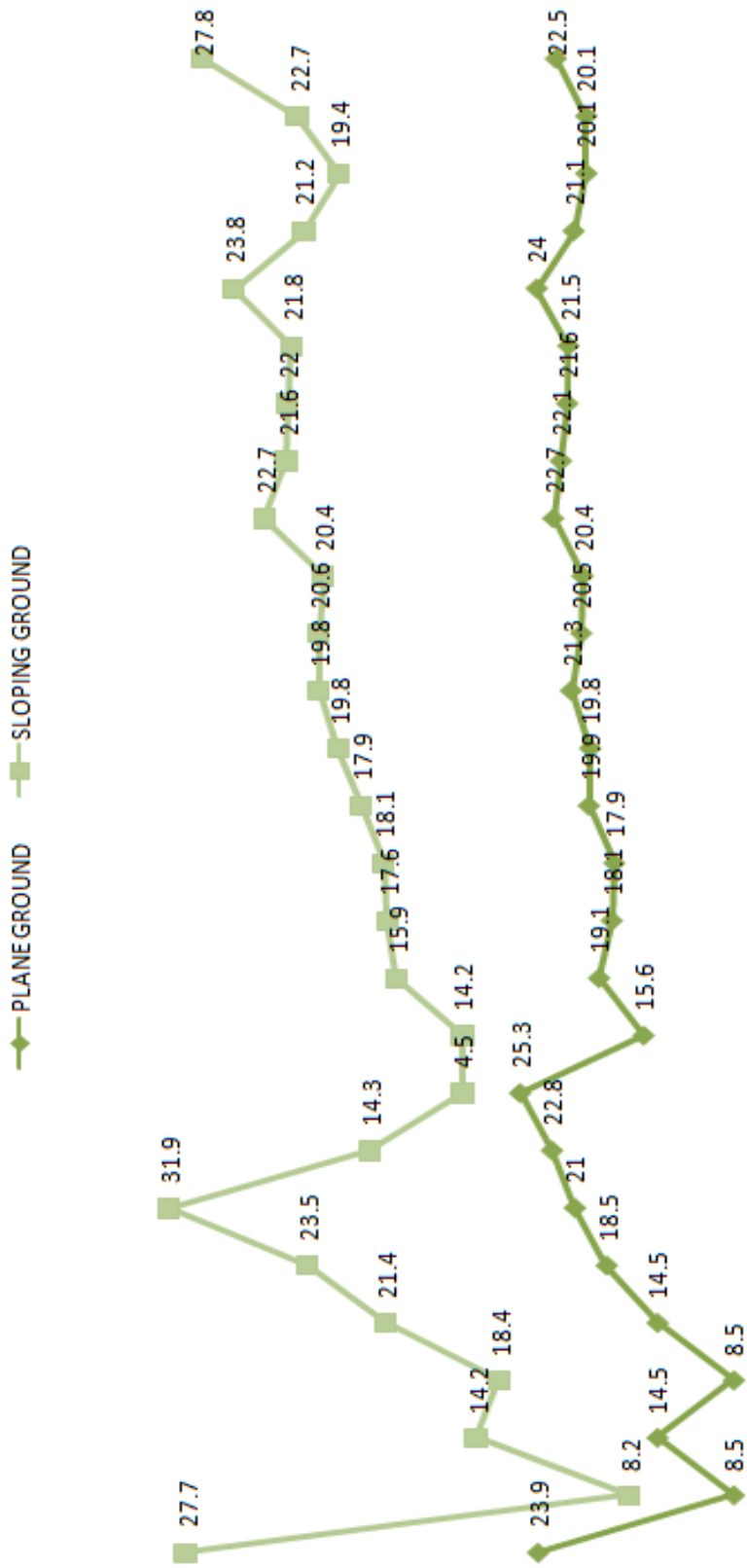
MAXIMUM TENSILE STRESS



MAXIMUM COMPRESSIVE STRESS



MAXIMUM COMPRESSIVE STRESS



PHASE-3

Under the **PHASE 3** a detailed design of a frame has been carried out with the design aid of IS456 and IS 13920:1993

This 3rd phase can be again broadly divided into following:-

3.1) design of an flexural member.

3.2) design of an exterior column.

3.3) design of an interior column.

To illustrate the design of a sub-frame a flexural member with maximum bending moment has been carried out .

General specification

- the member is designed according to IS 456:2000
- building > 3 storey height, minimum grade of concrete M20 – we used M30
- steel reinforcement of FE 415 used.

3.1)Design of the flexural member[2]

General

1) Factored axial stress less than $0.1 f_{ck}$

2) The member should preferably have a width to depth ratio of more 0.3

Width/depth=400/500=0.8 > 0.3 ,hence ok

3) Width should not be less then 200mm . But we provided width of 400 mm which is ok.

4) depth should not be greater than 0.25(clear span) i.e.(5000- 400)= 4600mm.

Longitudinal reinforcement (node 16)

We will find the reinforcement due to a sagging moment of 455 kN-m.

Assuming 25 mm dia bars with 25 mm clear cover

Effective depth(d)= 500-25-(25/2)=465mm

From table D of SP 16 : $1980 M_{u,lim}/bd^2 = 4.14$ (for M-30 and FE 415)

$M_{u,lim} = 4.14 \times 400 \times 465^2 = 358.1 \text{ kN-m.}$

Actual moment 455 kN-m is greater than 358.1 kN-m ,so we go for the doubly reinforced section.

Reinforcement from table 50 of SP 16: 1890

$$M_u/bd^2 = 5.26$$

$$d^1/d = 0.08$$

$$P_t (\text{bottom}) = 1.7028$$

$$P_c (\text{top}) = 0.784$$

Reinforcement due to the sagging moment

328.98kN-m is the design hogging moment which is not greater than 358.1kN-m ,so we go for the singly reinforced section.

$$M_u/bd^2 = 3.80$$

From table 4 we have,

$$P_t (\text{top})=1.282$$

now required reinforcement is the maximum of 0.784 and 1.282,so finally we have

Reinforcement due to the hogging moment

$$P_t (\text{top}) = 1.282$$

$$P_c (\text{bottom}) = 1.7028$$

$$\text{Reinforcement at top (A}_t\text{)}=1.282 \times 400 \times 465 = 2384 \text{ mm}^2.$$

$$\text{Reinforcement at the bottom}=1.7028 \times 400 \times 465 = 3167.20 \text{ mm}^2.$$

Checks

1) the top and bottom reinforcements should atleast contain 2 bars which is the case here.

2) tension steel ratio $p_{min} \leq 0.24 (f_{ck}/f_y)^{1/2} = 0.258$ but we have 1.7028. hence ok

Longitudinal reinforcement(node 34)

We will find the reinforcement due to a sagging moment of 235 kN-m.

Assuming 25 mm dia bars with 25 mm clear cover

$$\text{Effective depth}(d)= 500-25-(25/2)=465\text{mm}$$

From table D of SP 16 :1980 $M_{u,lim}/bd^2 = 4.14$ (for M-30 and FE 415)

$$M_{u,lim} = 4.14 \times 400 \times 465^2 = 358.1 \text{ kN-m.}$$

Actual moment 235 kN-m is less than 358.1 kN-m ,so we go for the singly reinforced section.

Reinforcement from table 4 SP 16: 1890

$$M_u/bd^2 = 2.717, d^1/d = 0.08$$

$$P_t \text{ (bottom)} = 0.80$$

Reinforcement due to the sagging moment

349 kN-m is the design hogging moment which is not greater than 358.1kN-m ,so we go for the singly reinforced section.

$$M_u/bd^2 = 4.03$$

From table 4 of SP 16 , we have

$$P_t \text{ (top)} = 1.391$$

so finally we have

$$P_t \text{ (top)} = 1.391$$

$$P_c \text{ (bottom)} = 0.80$$

$$\text{Reinforcement at top} = 1.391 \times 400 \times 465 = 2587 \text{ mm}^2.$$

$$\text{Reinforcement at the bottom} = 0.808 \times 400 \times 465 = 1503 \text{ mm}^2.$$

Checks

1) the top and bottom reinforcements should atleast contain 2 bars which is the case here.

2) tension steel ratio $p_{min} \leq 0.24 (f_{ck}/f_y)^{1/2} = 0.258$ but we have 0.808 hence ok

Shear reinforcement requirement

Shear force under consideration will be the maximum of the :-

1) Calculated shear force (V = 375)

2) Shear force due to the formation of the plastic hinges. At both the ends of the beam.

At node no 16

$$P_t = 3216/(400 \times 465) = 2.31\% \text{ (at top)}$$

$$M_{u,lim} / bd^2 = 6.9 \text{ (} P_t = 2.31, d'/d = 0.08 \text{)}$$

$$M_{u,lim} = 6.9 \times 400 \times 465^2 = 596 \text{ kN-m (maximum hogging moment)}$$

$$P_t = 2450/(400 \times 465) = 1.32\% \text{ (at bottom)}$$

$$P_t = 8.1 \text{ (} 1.32\% , d'/d = 0.08 \text{)}$$

$$M_{u,lim} = 8.1 \times 400 \times 465^2 = 700.56 \text{ kN-m (maximum sagging moment)}$$

At node 34

$$P_t = 1960/(400 \times 465) = 1.05\% \text{ (at bottom)}$$

$$M_{u,lim} / bd^2 = 7.3 \text{ (} P_t = 1.05\% , d'/d = 0.08 \text{)}$$

$$M_{u,lim} = 7.3 \times 400 \times 465^2 = 631 \text{ kN-m (maximum sagging moment)}$$

$$P_t = 2613/(400 \times 465) = 1.40\% \text{ (at top)}$$

$$M_{u,lim} / bd^2 = 4.15 \text{ (} P_t = 1.05\% , d'/d = 0.08 \text{)}$$

$$M_{u,lim} = 4.15 \times 400 \times 465^2 = 358 \text{ kN-m (maximum hogging moment)}$$

$$V_{34}^{D+L} = V_{16}^{D+L} = 1.2 \times (30 + 20) = 60$$

for sway to right:

$$V_{u,a} = V_a^{D+L} - 1.4 \left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bs}}{L_{AB}} \right]$$

$$\text{and } V_{u,b} = V_b^{D+L} + 1.4 \left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bs}}{L_{AB}} \right]$$

for sway to left:

$$V_{u,a} = V_a^{D+L} + 1.4 \left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}} \right]$$

$$\text{and } V_{u,b} = V_b^{D+L} - 1.4 \left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}} \right]$$

For sway to right

$$V_{u,34}=60-1.4[631+596]/4.6 = -313\text{kN}$$

$$V_{u,16}=60+1.4[631+596]/4.6 = 433$$

For sway to left

$$V_{u,34}=60+1.4[358+700.56]/4.6 = -382\text{ kN}.$$

$$V_{u,16}=60-1.4[358+700.56]/4.6 = -262\text{ kN}$$

The minimum percentage of steel used is = 1.05%

$$\tau_c = 0.66\text{ N/mm}^2.$$

$$\tau_v = 433/(400 \times 465) = 2.32\text{ N/mm}^2.$$

$$\tau_{1c,\text{max}} \text{ for M 30} = 3.5\text{ N/mm}^2.$$

$$V_{us} = V_u - \tau_c bd = 433 - 122 = 310\text{ N/mm}^2$$

We adopt 8 mm two legged stirrups

$$A_{sv} = 100.52\text{ mm}^2$$

S_{max} is minimum of

$$\text{a) } d/4 = 465/4 = 116$$

$$\text{b) } 8d_{\text{min}} = 8 \times 25 = 200$$

$$\text{c) } S = .87 \times 415 \times 100.5 \times 465 / (310 \times 1000) = 54.42 = 60\text{ mm}.$$

So we provide stirrups @ 60 mm c/c.

3.2)Design of exterior column[2]

In this example the columns of the ground floor are designed for illustrations. The exterior columns no 1 is designed for the forces based on maximum interaction ratio (1 in this case)

- We have size of the column 400mm x 500 mm
- Concrete mix M 30
- Vertical reinforcement Fe 415
- Axial load 1160 kN
- Moment from load 229 kN

The general requirement of the column for the ductility will follow IS 13920:1993 and vertical reinforcement of the column is designed according to IS 456:2000. The transverse and the special confinement reinforcement will be designed by following the IS 13920:1993 and IS 456:2000.

General (Column subjected to bending and axial load)

- IS 13920:1993 will be applicable if the axial stress $> 0.1 f_{ck}$.
 $1160 \times 1000 / (400 \times 500) = 5.8 > 0.1 f_{ck} = 3.$
- Minimum dimension of the member ≥ 250 and we have taken 400 which is ok.
- Shortest cross section dimension / perpendicular dimension ≥ 0.4 and we have the same ratio as 0.8

Vertical (longitudinal) reinforcement

Assume 20 mm ϕ with 40 mm cover ($d_1 = 40 + 10 = 50$ mm, $d_1 / D = 50 / 500 = 0.1$)

From chart 44 SP 16: 1980 ($d_1 / D = 0.1$, 415 N/mm^2)

$$P_u / f_{ck} b D = 1160 \times 1000 / (30 \times 400 \times 500) = 0.19$$

$$M_u / f_{ck} b D^2 = 0.075$$

Reinforcement on four sides from chart 44 SP 16: 1980

$$P / f_{ck} = 0.045, \text{ reinforcement in } \% = 0.045 \times 30 = 1.35 \%$$

$$A_s = p_b d / 100 = 1.35 \times 400 \times 500 / 100 = 2700 \text{ mm}^2 \text{ (8 @ 22mm } \phi \text{ bars of area 3040 mm}^2 \text{)}$$

Some important notes:-

- Lap splices only at the central half of the member
- Hoops over the entire splice length at a spacing < 150

- Not more than 50 % of the bars are spliced at one section.

Transverse reinforcement

Some important points:-

- Hoop requirement as per fig 7 in IS 13920:1993
- If the length of the hoop > 300 mm a cross tie shall be provided as shown in fig
- Hoop spacing should not exceed half the least lateral dimension of the column i.e.
 $400/2 = 200 \text{ mm}$

The design shear force for the column shall be the maximum of the the following:-

a) calculated shear force as per the analysis which is 53.29 kN.

b) factored shear force as given by $V_u = 1.4 [M_{u,lim}^{bL} + M_{u,lim}^{bR} / h_{st}]$

where $M_{u,lim}^{bL}$ and $M_{u,lim}^{bR}$ are the moments of opposite sign of beams framing in to the column from opposite faces and h_{st} is the storey height.

moment of resistance of the beam at node 2 ,assuming all other beams of the same floor is designed for the same critical design conditions as that of beam 54, we have

At node 2

$$P_t = 1960 / (400 \times 465) = 1.05\% \text{ (at bottom)}$$

$$M_{u,lim} / bd^2 = 7.3 (P_t = 1.05\%, d'/d = 0.08)$$

$$M_{u,lim} = 7.3 \times 400 \times 465^2 = 631 \text{ kN-m (maximum sagging moment)}$$

$$P_t = 2613 / (400 \times 465) = 1.40\% \text{ (at top)}$$

$$M_{u,lim} / bd^2 = 4.15 (P_t = 1.05\%, d'/d = 0.08)$$

$$M_{u,lim} = 4.15 \times 400 \times 465^2 = 358 \text{ kN-m (maximum hogging moment)}$$

$$V_u = 1.4 [631/6] = 147.23 \text{ kN}$$

$$\phi = 1 + 3 P_t / A_{1g} f_{1ck} = 1 + (3 \times 1160 \times 1000) / [(400 \times 500 - 3040) \times 30] = 1.5890 > 1.5$$

So we use $\phi = 1.5$

$$\text{Now corrected } \tau_c = 1.5 \times 0.724 = 1.086 \text{ N/mm}^2.$$

$$V_c = \tau_c bd = 1.086 \times 400 \times 500 = 217.2 \text{ kN.}$$

Thus nominal shear reinforcement is to be provided in accordance to IS 456.

We use 8 mm Φ , two legged stirrups of area 100.5 mm².

For minimum stirrups we have

$$S_v \leq A_{sv} \cdot 0.87 f_y / 0.4 \text{ i.e. } \leq 225$$

The spacing shall be lesser of the

a) $0.75 d = 0.75 \times 475 = 356$

b) 256 as calculated.

So we provide 8 mm phi two legged stirrups @ 225 mm. c/c

Special confining reinforcement

Special confining reinforcement is to be provided over a length of l_0 towards the mid span of the column

$$l_0 \geq [\text{depth of the member} = 500 \text{ mm}]$$

$$[1/6 \text{ of the clear span which is 1 m in this case}]$$

$$[450 \text{ mm}]$$

The spacings of the hoop shall not exceed

$$S_{\max} < [1/4 (\text{minimum member dimensions}) = 100 \text{ in this case}]$$

$$[\text{Should not be less than 75}]$$

$$[\text{Should not be greater than 100}]$$

Minimum area of cross section of the bar forming hoop is

$$A_{sh} = 0.18 s h f_{ck} / f_y (A_g / A_k - 1)$$

We use $s = 100$ mm from above and $h = 400$

$$\text{So we have } A_{sh} = 130 \text{ mm}^2.$$

Using 10 mm dia bar (78.53 mm²) at a spacing of $100 \times 78.53 / 130 = 60$ mm i.e . @ 60 mm c/c

3.3)Design of interior column/2]

In this example the columns of the ground floor are designed for illustrations. The exterior column no 28 is designed for the forces based on maximum interaction ratio (1 in this case)

- We have size of the column 400 mm x 500 mm
- Concrete mix M 30
- Vertical reinforcement Fe 415
- Axial load 2220 kN
- Moment from load 398 kN

The general requirement of the column for the ductility will follow IS 13920:1993 and vertical reinforcement of the column is designed according to IS 456:2000. The transverse and the special confinement reinforcement will be designed by following the IS 13920:1993 and IS 456:2000.

General(Column subjected to bending and axial load)

- IS 13920:1993 will be applicable if the axial stress $> 0.1 f_{ck}$.
 $2260 \times 1000 / (400 \times 500) = 11.3 > 0.1 f_{ck} = 3.$
- Minimum dimension of the member ≥ 250 and we have taken 400 which is ok.
- Shortest cross section dimension / perpendicular dimension ≥ 0.4 and we have the same ratio as 0.8

Vertical (longitudinal) reinforcement

Assume 20 mm Φ with 40 mm cover ($d_1 = 40 + 10 = 50$ mm, $d_1 / d = 50 / 500 = 0.1$)

From chart 44 SP 16: 1980 ($d_1 / d = 0.1$, 415 N/mm^2)

$$P_u / f_{ck} b D = 2220 \times 1000 / (30 \times 400 \times 500) = 0.36$$

$$M_u / f_{ck} b D^2 = 0.1327.$$

Reinforcement on four sides from chart 44 SP 16: 1980

$$P / f_{ck} = 0.08 \%, \text{ reinforcement in } \% = 0.08 \times 30 = 2.4 \%$$

$$A_s = p b d / 100 = 2.4 \times 400 \times 500 / 100 = 4800 \text{ mm}^2 \text{ (13 @ 22mm } \Phi \text{ bars of area 4940mm}^2 \text{)}$$

Some important notes:-

- Lap splices only at the central half of the member
- Hoops over the entire splice length at a spacing < 150
- Not more than 50 % of the bars are spliced at one section.

Transverse reinforcement

Some important points:-

- Hoop requirement as per fig 7 on IS 13920:1993
- If the length of the hoop > 300 mm a cross tie shall be provided as shown in fig
- Hoop spacing should not exceed half the least lateral dimension of the column i.e.
 $400/2 = 200 \text{ mm}$

The design shear force for the column shall be the maximum of the the following:-

a) calculated shear force as per the analysis which is 154 kN.

b) factored shear force as given by $V_u = 1.4 [M_{u,lim}^{bL} + M_{u,lim}^{bR} / h_{st}]$

where $M_{u,lim}^{bL}$ and $M_{u,lim}^{bR}$ are the moments of opposite sign of beams framing in to the column from opposite faces and h_{st} is the storey height.

moment of resistance of the beam at node 2 ,assuming all other beams of the same floor is designed for the same critical design conditions as that of beam 54, we have

At node 34

$$P_t = 1960 / (400 \times 465) = 1.05\% \text{ (at bottom)}$$

$$M_{u,lim} / bd^2 = 7.3 (P_t = 1.05\%, d'/d = 0.08)$$

$$M_{u,lim} = 7.3 \times 400 \times 465^2 = 631 \text{ kN-m (maximum sagging moment)}$$

$$P_t = 2613 / (400 \times 465) = 1.40\% \text{ (at top)}$$

$$M_{u,lim} / bd^2 = 4.15 (P_t = 1.05\%, d'/d = 0.08)$$

$$M_{u,lim} = 4.15 \times 400 \times 465^2 = 358 \text{ kN-m (maximum hogging moment)}$$

$$V_u = 1.4 [631 + 358 / 5.5] = 251.7 \text{ kN}$$

$$\phi = 1 + 3 P_t / A_{1g} f_{1ck} = 1 + (3 \times 2220) / [(400 \times 500 - 4940) \times 30] = 2.13 > 1.5$$

So we use $\phi = 1.5$

$$\text{Now corrected } \tau_c = 1.5 \times 0.87 = 1.305 \text{ N/mm}^2.$$

$$V_c = \tau_c bd = 1.305 \times 400 \times 500 = 261 \text{ kN} > 251 \text{ kN}$$

Thus nominal shear reinforcement is to be provided in accordance to IS 456.

We use 8 mm dia two legged stirrups of area 100.5 mm².

For minimum stirrups we have

$$S_v \leq A_{sv} \quad 0.87f_y / 0.4 \text{ i.e. } \leq 225$$

The spacing shall be lesser of the

a) $0.75 d = 0.75 \times 475 = 356$

b) 256 as calculated.

So we provide 8 mm phi two legged stirrups @ 225 mm. c/c

Special confining reinforcement

Special confining reinforcement is to be provided over a length of l_0 towards the mid span of the column

$$l_0 \geq [\text{depth of the member} = 500 \text{ mm}]$$

$$[1/6 \text{ of the clear span which is } 1 \text{ m in this case}]$$

$$[450 \text{ mm}]$$

The spacing of the hoop shall not exceed

$$S_{\max} < [1/4 (\text{minimum member dimensions}) = 100 \text{ in this case}]$$

$$[\text{Should not be less than } 75]$$

$$[\text{Should not be greater than } 100]$$

Minimum area of cross section of the bar forming hoop is

$$A_{sh} = 0.18 sh f_{ck} / f_y (A_g / A_k - 1)$$

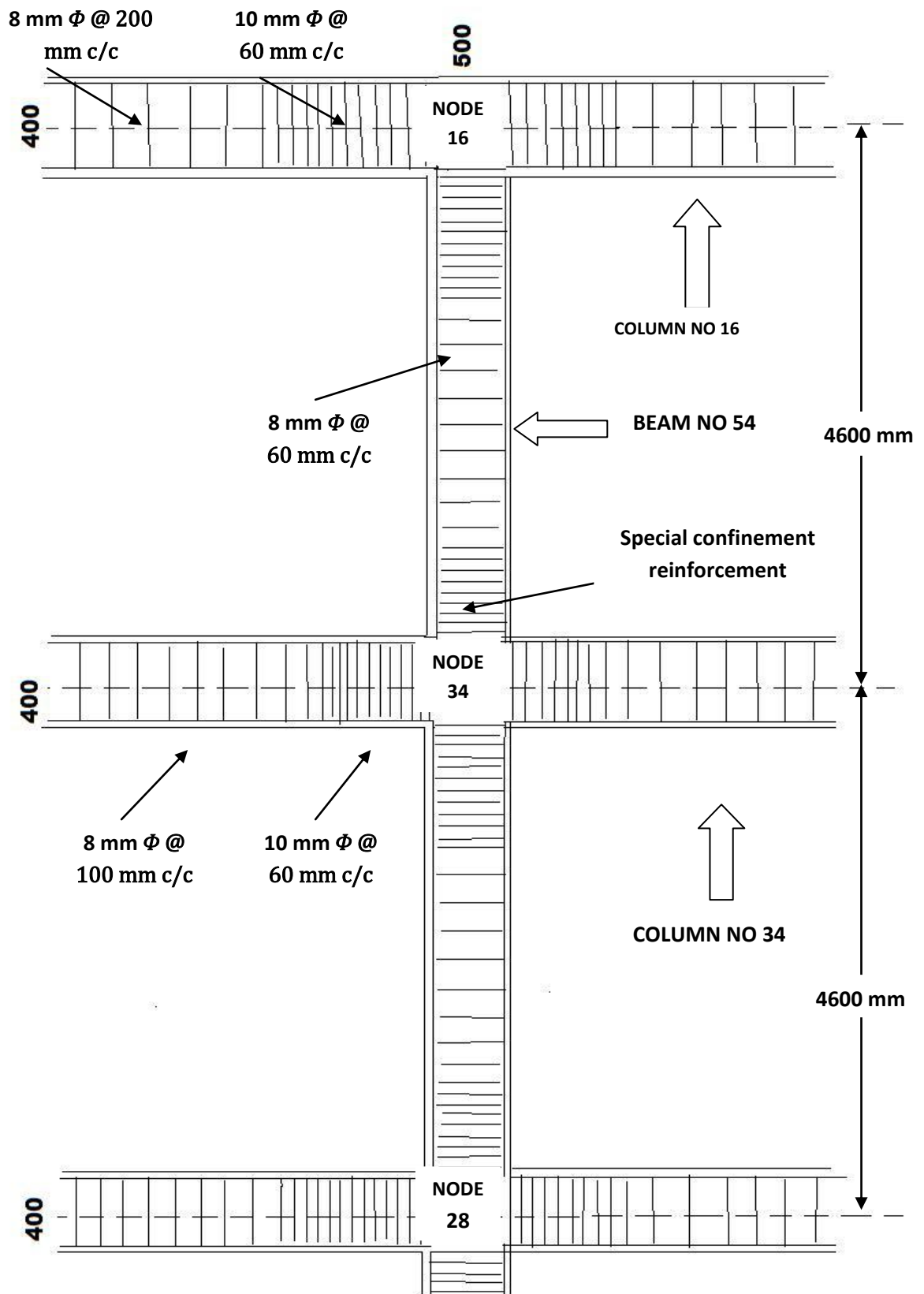
$$A_{sh} = 0.18 sh f_{ck} / f_y (A_g / A_k - 1)$$

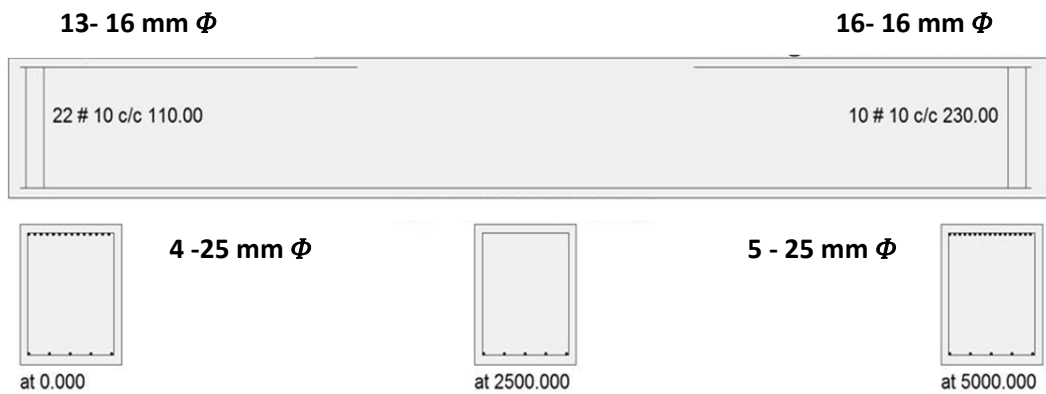
We use $s = 100 \text{ mm}$ from above and $h = 400$

So we have $A_{sh} = 130 \text{ mm}^2$.

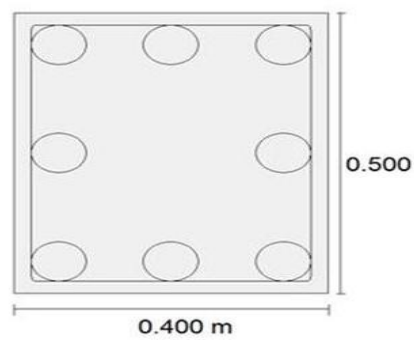
Using 10 mm dia bar (78.53 mm^2) at a spacing of $100 \times 78.53 / 130 = 60 \text{ mm}$ i.e . @ 60 mm c/c

DUCTILITY DETAILING:-

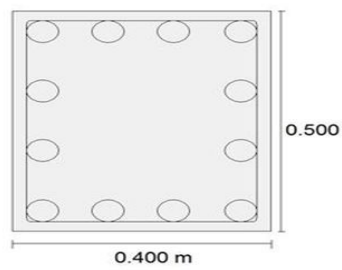




BEAM NO 54



COLUMN 1



COLUMN NO 34

CONCLUSION

The tasks of providing full seismic safety for the residents inhabiting the most earthquake-prone regions are far from being solved. However in present time we have new regulations in place for construction that greatly contribute to earthquake disaster mitigation and are being applied in accordance with world practice. [8]* [4]*

In the regulations adopted for implementation in India the following factors have been found to be critically important in the design and construction of seismic resistant buildings:

- sites selection for construction that are the most favourable in terms of the frequency of occurrence and the likely severity of ground shaking and ground failure;
- high quality of construction to be provided conforming to related IS codes such as IS 1893 , IS 13920 to ensure good performance during future earthquakes.
- To implement the design of building elements and joints between them in accordance with analysis .i.e. ductility design should be done.
- structural-spatial solutions should be applied that provide symmetry and regularity in the distribution of mass and stiffness in plan and in elevation.
- Whereas such the situations demands irregularity maximum effort should be given to done away with the harmful effects like that of “ SHORT COLUMN EFFECT”

Researchers indicate that compliance with the above-mentioned requirements will contribute significantly to disaster mitigation, regardless of the intensity of the seismic loads and specific features of the earthquakes. These modifications in construction and design can be introduced which as a result has increase seismic reliability of the buildings and seismic safety for human life.

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